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SOIL MECHANICS AND FOUNDATIONS DIVISION

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SYMPOSIUM ON GROUTING: GROUTING OF GRANULAR MATERIALS[a]

By John C. King, F. ASCE,[1] and Edward G. W. Bush, M. ASCE[2]

FOREWORD

This paper is one of several papers that constitute the Symposium on Grouting that occupied two sessions of the June, 1960 meeting of ASCE in Reno, Nevada. Credit for organizing this symposium program goes to a Special Grouting Symposium Committee consisting of Stanley J. Johnson, F. ASCE, Chairman, T. William Lambe, A.M. ASCE, and W. K. Seaman, F. ASCE.

It is believed that these several papers constitute a significant contribution to engineering practice in the art and science of stabilizing foundation materials by grouting. This is a field in which there appears to be increased interest among foundation engineers and one that is expected to become more important as time goes on and the need to build structures on difficult soil conditions increases.

The papers of the symposium are concerned with cement, clay, and chemical grouting of seams, cavities, and granular materials. The papers illustrate the increased use of chemicals for grouting purposes and also their use with more conventional grouting materials such as clay and cement; grouting of overburdened materials that is rapidly becoming a more definite art and science and that, to some degree at least, is susceptible to laboratory studies; the grouting of a cut-off in granular materials to prevent underseepage beneath a major hydroelectric project (perhaps the first application of this type of grouting in the United States); and the grouting of subsurface voids and solution channels in limestone areas subject to large subsidence.

Note.—Discussion open until September 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings, of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

[a] Presented at the June 21, 1960 ASCE Convention at Reno, Nev.

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Organized in 1952, the Grouting Committee with Task Committees on Cement, Clay, and Chemical Grouting has sponsored groups of papers or symposia occupying the sessions of meetings of the Society in 1954 and 1956, as well as 1960. Papers on grouting that have been published in the Journal of the Soil Mechanics and Foundations Division are as follows: Nos. 1145, 1204, and 1544 to 1552 inclusive. The report of the Task Committee on Chemical Grouting appears as Paper 1426 in the November 1957 Journal. A draft of a report on clay grouting has been prepared by the Task Committee on Clay Grouting; and the report on cement grouting prepared by the Task Committee on Cement Grouting is substantially in its final form.

The Administrative Committee on Grouting takes this opportunity of expressing its appreciation to the task committee for the excellent manner in which they have carried out their assignments and to the individuals who, as authors of papers, have made significant contributions covering the practices of grouting in various fields.

Raymond E. Davis, F. ASCE, Chmn., Admin. Committee on Grouting, Prof. of Civ. Engrg., Emeritus, Univ. of California, Berkeley, Calif.

SYNOPSIS

Success in grouting granular materials with solid suspension grouts depends on the proper choice of both materials and techniques. This paper presents information relating in-place grain size with pore dimensions and suitable grout particle diameter and describes both materials and techniques successfully used in a variety of types of foundation materials.

INTRODUCTION

The gradation and location of the various clay, sand, and gravel layers to be tightened or stabilized by grouting are too frequently imperfectly known, even on the larger jobs. As a result, when grouting appears to be indicated owners and consultants tend to rely on the grouting contractor, not only for the execution of the work, but for much of the specialized consulting and test work also. The good grouting contractor is a specialist who has, and relies on, a wide experience in many kinds of related grouting work—in soils, rock and cracked masonry and the use of cement, clays, and so forth, with or without additives, as well as chemicals with or without solids in suspension.

An air of secrecy has surrounded the operation of drilling and grouting, with each company guarding its trade secrets, particular techniques, and patent formulas to which it attributes its success; for the lack of free knowledge throughout the industry, each contractor has hoped to oblige owners and consultants to turn to him, rather than to his competitors. In brief, grouting has been handled almost as an art, as opposed to the science it should much more nearly be.

A number of comprehensive and authoritative papers on cement, clay and chemical grouting published under the sponsorship of the Committee on

Grouting of the Soil Mechanics and Foundations Division is the start of a major breakthrough in the dissemination of knowledge to the consulting engineer about the unspectacular business of grouting. The writers hope to widen the breach by relating practical information on grain size of suspended materials with pore size in granular soils, by presenting specific information on grouting materials, and to describe some techniques that have successfully placed many of these materials in the ground.

OBJECTIVES

Grouting of granular materials is usually done to arrest or reduce water movement, to strengthen the material for the purpose of increasing bearing capacity or reducing settlement under existing loads, or both of these functions. Grouting is also done to increase shearing resistance for stability against lateral movement.

The term granular materials is used here in its broadest sense, and includes all soils from clay through fine sand to coarse gravels, boulders, and talus. The methods, materials and results obtainable in such a spread of grain sizes vary greatly. In clays water movement is extremely slow even under high pressure. In such material grouts can only displace the grains, penetrating planes of weakness to form lenses of grout or radially displacing the material to form bulbs. The material to be used in grouts for this purpose is cement, or combinations of cement and other solids that will form layers or bulbs of hardened material. The strength that such grouting will add is questionable.

When the grain size is such that water will move with relative ease (in finite construction time) chemical solution grouts may be used to fill voids, displacing any water that may be present, or solid-suspension grouts may be pumped, that although they will not flow between the grains, will form bulbs and lenses and displace them locally causing consolidation by driving out water. Combinations of sand and silt with small proportions of clay are examples of materials that may be so consolidated.

As the grain size increases to coarse sand and fine gravel, solid-suspension grouts can be pumped into the voids to consolidate and solidify the materials. Finally, in masses with gravel or rock particles large enough to provide connected voids of 1/4 in. minimum dimension, a rich sanded grout may be pumped to provide high strength, or a lean sanded grout for economical watertightness and stability. (Where void dimensions exceed 3/4 in. as in 3 1/2 in. or larger stone, lean grout with suspended pea gravel may be used.)

Where water is moving through the larger voids at sufficient velocity to wash away cement and sand, large particle suspensions may be used to deliberately choke the passages and provide a filter that will catch and hold the finer materials.

INFLUENCE OF PORE SIZE ON GROUTING MATERIALS

Data on sieve analysis, blow count, water table, and boulder or rock levels are most often available from soil sampling tests carried out during the planning stage of a job. It is the job of the engineer to interpret this data and especially to translate the sieve analysis and blow count into meaningful terms

of pore size and groutability ratios. The key to the choice of grouting materials lies in the determination of the effective pore size of the sand or gravel to be grouted. It can be demonstrated that there is a definite relationship between the size of the grains in a system of packed rock particles and the effective pore diameter or "critical ratio of entrance."[3]

Mathematical examination[3,4,5] of a system of spherical particles indicates that the critical ratio of entrance or net pore diameter may vary from 15% of the diameter of the sphere at 26% void ratio to 41% at 48% voids, as shown in Fig. 1.

It appears to be generally agreed that no serious errors are introduced by assuming the sand grains to be spherical. Most naturally occurring sands

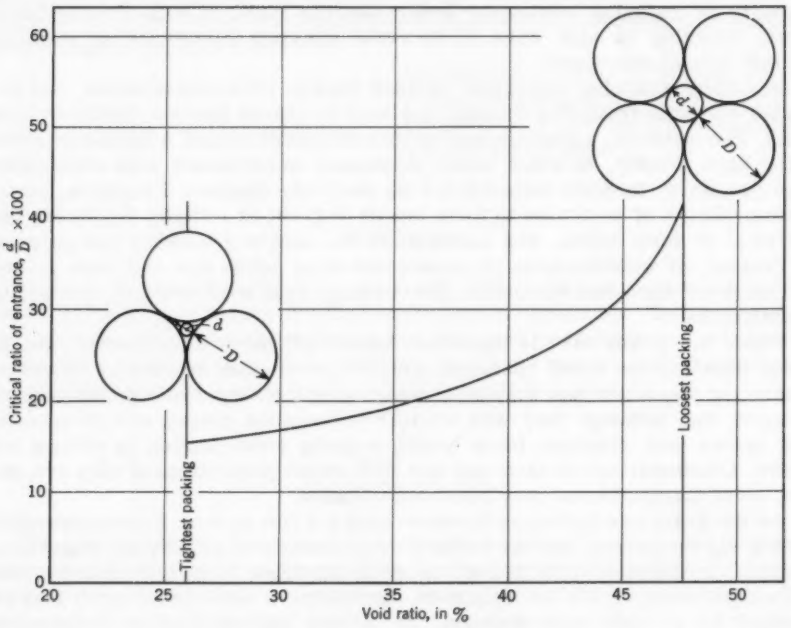

FIG. 1.—SYSTEMATIC PACKING OF SPHERES

or gravels in old river beds are sufficiently abraded to be considered as spheroids for this purpose, and will be found to have an outside range of void ratios in place of from 30% to 40%, with a far larger number in the range of 35% to 38% due to tighter packing resulting from a mixture of sizes. In Fig. 1,

---

3 "Experimental Study of Porosity and Permeability of Clastic Sediments," by H. J. Frazer, Journal of Geology, Vol. 43, No. 8, 1935.

4 "Systematic Packing of Spheres - With Particular Relation to Porosity and Permeability," by I. C. Graton and H. J. Frazer, Journal of Geology, Vol. 43, No. 8, 1935.

5 "Volume, Shape and Roundness of Quartz Particles," by H. Wadell, Journal of Geology, Vol. 43, No. 3.

it is seen that this range of porosity corresponds to a net pore diameter of 19.5% to 21.5% of the diameter of the large spheres.

In order to verify the validity of this relationship between grain size and pore size for natural sands and gravels, the writers carried out a series of tests on single-sized aggregates to determine critical pore diameter ratio and the shape of the ratio spectrum. Standard 6 in.-by-12 in. cylinder molds were filled with three sizes of aggregate, 1/2 in. to 3/4 in.; No. 4 to 3/8 in. and No. 8 to No. 4, each well shaken down and grouted with a neat cement grout. While the grout was still reasonably soft the cylinders were carefully broken into two halves, the soft grout dug out of all the visible pore spaces with a dentist's cavity probe, and the various pore diameters measured and counted. The results of the measurements on the 1/2 in. to 3/4 in. stone are presented in Fig. 2, and are the basis of the pore diameter ratios used in Table 1.

Photographs of some of the test specimens are given in Fig. 3. Examination of the two smaller sizes of stone generally confirm the findings on the large

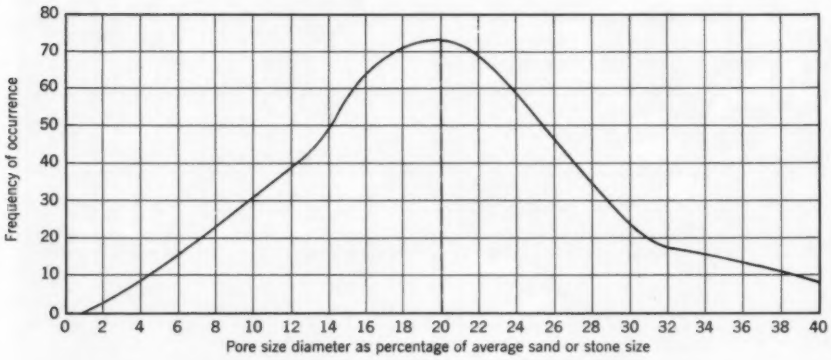

FIG. 2.—PORE SIZE DISTRIBUTION FOR 1/2 IN. TO 3/4 IN. STONE

size, but with a lesser degree of accuracy due to the methods used. It was observed that whereas there were a fair number of pores at higher ratios than 20%, all of those above 30% were occluded at some point by smaller pores, and that a very large proportion of those over 25% were similarly blocked off. By taking the design figure at 20%, the existence of the largest number of inter-connecting pores of that size is assured. The aggregates used in these tests were crushed gravel and pea gravel. Obviously different types of aggregate and mixed gradings are going to give widely varying pore ratios and shape of the "spectrum," and it is suggested that more detailed investigation of this subject is a worthwhile field for research.

The groutability ratios of Table 1 are arrived at by combining the 20% pore diameter ratio found above with the premise that for effective injection to take place, the pore size should be at least three times the effective maximum grain size of the cementing materials. This stems from the assumption that two grains of cement or sand wedged across the opening of a pore will

TABLE 1.—GROUTABILITY RATIOS BASED ON N = 13

Grouted Materials				Grouting Materials D85/ and /D95 Size Microns												
D15 or D10 Sieve No.	Average Sieve Size Opening Microns	Pore Size Sieve Opening ÷ 5 Microns	Pore Size ÷ 3 Microns	Type I Cement	Type II	Type III	Type III Scalped	Fly Ash	Ground Slag	200 Mesh Silica Flour	Clays	Bentonite	Asphalt	Special Asphalt	Sands and Silts	Sieve No. of Sands and Silts
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
325	44	8.8	2.9									1.5/		1.8/2.2		
200	74	14.8	4.9													
100	149	29.8	10.0									/7	7/8.5			
50	297	59.4	19.8				17/		18/							
40	430	86	28.7			25/34	/26		/27		26/					
30	590	118	39.3		40/			33/		36/						
20	850	170	57		/52			52/47		/51	/50				44	325
16	1190	238	80	67/74											74	200
14	1420	284	95													
10	2000	400	133					/100								
8	2380	477	159												149	100
4	4760	952	317												297	50
3/8 in.	9520	1905	635												590	30
1/2 in.	12700	2540	848												850	20
5/8 in.	15900	3180	1060													
3/4 in.	19100	3820	1273												1190	16
1 in.	25400	5080	1700													
1 1/2in.	38100	7620	2530												2380	8
2 in.	50800	10150	3400													
3 in.	76200	15250	5080												4760	4

form a stable bridge, but that an arch formed by three grains will not be stable.

In other words, if the diameter of grouting particles is not over 1/5 x 1/3 = 1/15 of the particle diameter of the material to be grouted, the grout will pass freely through the void system.

Obviously, a few larger grouting particles and a few smaller openings will not choke the system. The Corps of Engineers Waterways Experiment Station, Vicksburg, Miss.,[8] proposed, on the basis of experimental data, that grouting could be successfully accomplished if the groutability ratio were

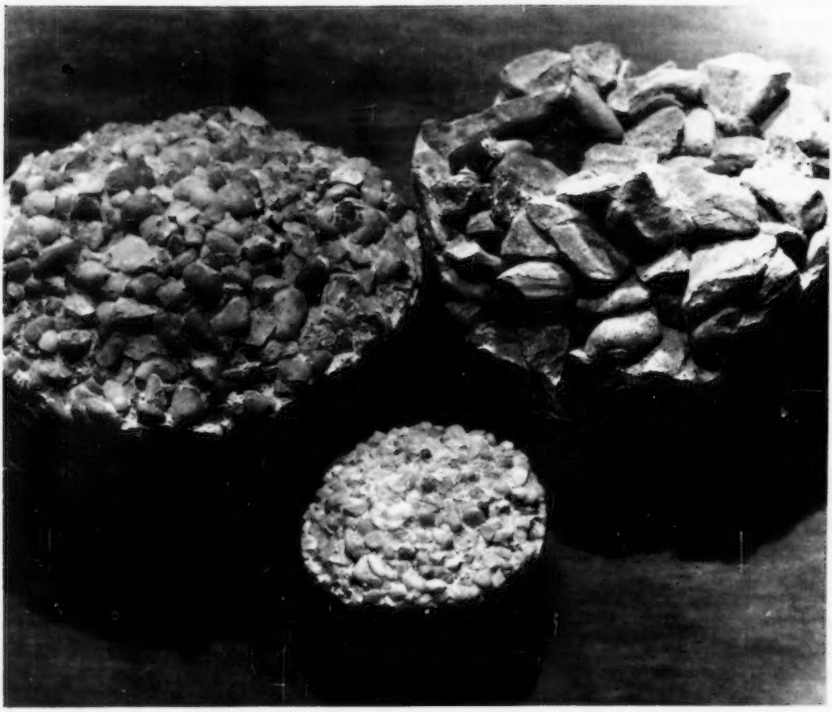

FIG. 3.—SPECIMENS FOR DETERMINATION OF PORE SIZE DIAMETER

greater than $N = 15 = D_{15}/D_{85}$ where D_{15} and D_{85} are the effective grain size diameters in microns of the indicated percentages passing (15% and 85%) of grouted and grouting materials, respectively. Thus a sand having 15% passing the No. 16 sieve is said to have a D_{15} size of 1190 microns, and a cement having 85% passing the No. 200 sieve has a D_{85} size of 74 microns. The groutability ratio for this combination is $N = 1190/74 = 16$.

It is not enough to rely solely on the D_{85} figure for the maximum size of the grouting material. The ratio $N = D_{15}/D_{85}$ may be satisfactory, for example if N = 16, but if the ratio D_{10}/D_{95} comes out about 8, the feasibility of

grouting is open to question, because when N approaches 6, it is known that filtering starts to take place. On Fig. 4 are plotted a number of successful grouting parameters, both as D_{15}/D_{85} and as D_{10}/D_{95}. It is seen that the D_{10}/D_{95} values are generally lower than the D_{15}/D_{85} but that they do not appear above the line N = 8.

In Fig. 4 the grain sizes for bentonite are derived from a wet analysis of a typical "ninety barrel minimum" material from Wyoming. Not all bentonites are as fine or high gelling as this, but it is a quality to be aimed at for the penetration of fine voids. The term "ninety barrel minimum" is one used in the well-drilling industry, and means that one ton of this bentonite when mixed with water under standard conditions will produce ninety barrels (of 45 gal each) of 15 centipoise mud, or 30 sec in a Marsh funnel.

In Fig. 5 are plotted some typical gradings of a coarse and fine fly ash and four cements, Type I, II, III, and III S, the last being a scalped Type III having 98.5% passing the 325 sieve. The 85% and 95% passing sizes are also indicated by the upper and lower limits of the different types of cement, fly ash, clay, slag, and asphalt on Fig. 5. The graphical representations are for particular samples, and as such are not valid for all sources of manufacture.

TABLE 2

Sieve Number (1)	Opening Microns (2)	Individual Percent Retained	
		Type II Cement (3)	Fly Ash (4)
50	297	0 -0.2	0.2-0.9
100	149	0 -0.1	0.9-1.3
200	74	1.3-4.3	2.5-6.2
Blaine fineness, in square centimeters per gram		3072	2795

There is no substitute for the actual testing of a particular cement, clay or other material under consideration for use. The coarse fly ash shown had 20% retained on a 325 sieve. Worse still, its D_{95} size of 100 microns is nearly twice its D_{85} size of 52 microns that will, therefore, halve the N value and justify its rejection for use in grouting a sand having more than 10% passing a No. 16 sieve.

A good example of such an unsuitable fly ash is detailed in a technical report[6] of the Corps of Engineers, from which the data for Table 2 was extracted. The Type II cement used in this test, that apparently had a grading about the same as the Type II of Fig. 5 giving it a D_{95} size of 50 microns, was successfully injected into a 254 micron (0.01 in.) fissure. The fly ash on the other hand had sufficient material retained on the No. 50 and No. 100 sieves (297 microns and 149 microns) to form a filter in the 254 micron opening. This made it impossible to inject it even when used in combination with the same Type II cement.

6 "Pressure Grouting Fine Fissures, Technical Report No. 6-437," Corps of Engrs., Waterways Experiment Sta., Vicksburg, Va.

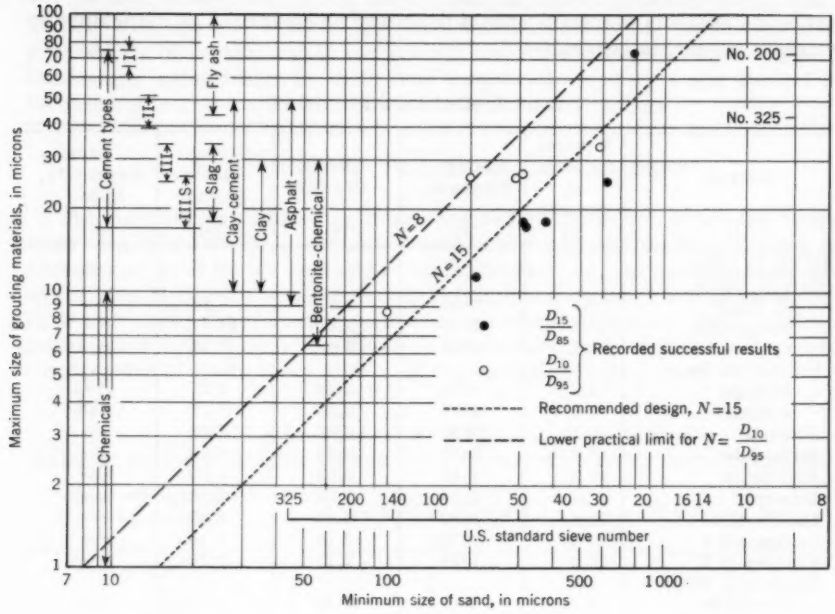

FIG. 4.—BENTONITE GRAIN SIZE FROM A WET ANALYSIS

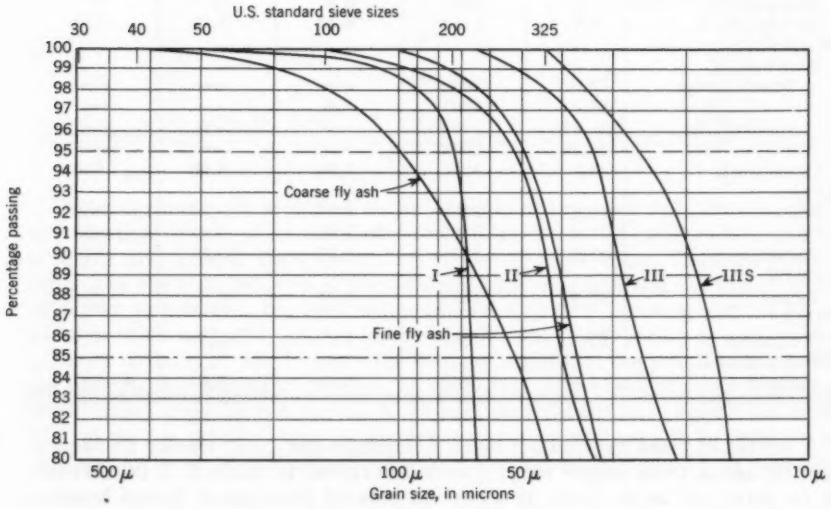

FIG. 5.—COMPARISON OF MAXIMUM SIZES OF SOME CEMENT TYPES

Not all fly ashes, of course are like the one in this illustration, and the use of it as an example of failure should in no way be construed as derogatory to fly ash or pozzolans generally.

TABLE 3.—DATA ON FLY ASHES

Source (1)	Percent Retained on No. 325 (2)	Blaine Fineness (3)	Specific Gravity (4)	Percent Loss on Ignition (5)	Lime Reactivity, in psi (6)
UNITED STATES					
Detroit, Mich.					
St. Clair	13.1	2930	2.50	3.96	---
Marysville	19.4	5120	2.23	11.0	---
Trenton	19.7	2940	2.47	3.12	---
Pittsburgh, Pa.					
Phillips	13.8	2850	2.34	2.21	1071
Elrama	3.0	4870	2.39	3.23	1000
Mitchell	15.6	2790	2.38	3.32	---
Springdale	15.8	2470	2.34	4.80	---
Burlington	11.2	2540	2.62	4.20	---
Seawaren	14.4	2100	2.63	3.33	---
Ludlowville	21.6	2080	2.50	2.07	---
Bridgeport I	18.4	2671	2.14	3.15	---
Bridgeport II	27.5	1836	2.18	3.76	---
Baltimore, Md.					
Riverside	19.2	5162	2.23	8.76	---
Philadelphia, Pa.					
Plymouth Meeting	39.5	---	---	12.70	---
Cleveland, Ohio	7.8	3660	2.39	6.68	---
Tarrytown, N. Y.	33.9	2360	---	9.25	---
Chicago, Ill.	8.0	3450	2.45	3.00	967
Montgomery, W. Va.	18.0	1990	2.26	0.58	---
Oswego, N. Y.	18.6	1755	2.38	2.42	776
ENGLAND					
Manchester					
Stuart Street	72.0	1000	---	---	---
	18.0	3205	---	---	---
Agecroft B	21.64	4028	---	---	---
	48.55	3042	---	---	---
Littlebrook	25.7	3380	2.06	4.56	790
JAPAN					
Ube	23.9	2500	1.92	0.40	550
Osaka	12.4	3720	2.03	1.02	669
Tobata	27.8	2530	1.95	1.11	427
TAIWAN					
Taipei	17.0	4410	2.27	6.56	705
TURKEY					
Zonguldak	53.1	1260	1.9	1.4	474
NEW ZEALAND					
Aukland	16.5	3840	2.03	2.61	693

A series of tests carried out by S. J. Rehmar and C. E. Peeler on twenty-eight fly ashes from widely varying sources is given in Table 3. It is interesting to note that while there is a certain general lowering of Blaine fineness with increase in percentage retained on the 325 sieve, both sets of figures should be taken into consideration when assessing a fly ash for use in grouting either sands or fine fissures.

The writers agree generally with the conclusions of the Corps of Engineers, Mayer, Kravitz, and others, and have incorporated known limitations of the groutability of various sands and gravels into the framework of pore size relationships established in their own investigations. Fig. 4 and Table 1 set forth suitable grouting combinations based on these findings.

IMPORTANCE OF BLEEDING AND EXPANSION

The importance of bleeding is emphasized by Table 4 in which the figures were obtained on neat cement grouts after setting for 4 hr, and are not necessarily good for all cements or methods of mixing, but are representative of the kind of results to be expected. In fact, because these mixes were made in a high speed laboratory mixer with a 6 min mixing time, field results of mixes made with ordinary paddle-type mixers are likely to give much greater bleeding due to inefficiency of mixing and curtailed mixing time.

TABLE 4.—BLEEDING OF NEAT CEMENT GROUTS

Water-Cement Ratio by Weight (1)	Percentage Bleeding (2)	Penetration Ratio (3)
0.40	0	0.4
0.55	2	1.0
0.65	10	4.0
0.75	12.5	5.8
0.90	20	9.0
1.00	25	11.5
1.15	30	16.0
1.50	40	30.0
1.85	50	65.0
2.25	60	90.0
2.70	70	160.0
3.12	80	270.0
3.60	90	480.0
(4.0) Extrapolated	(99)	(720.0)

It is obvious that bleeding water accumulating under individual particles will destroy the bond on the undersides of the particles[7] and reduce both the strength and lateral impermeability of the grouted mass. Because grouting often has the dual purpose of consolidation for strength and for the arresting of water movement, the two characteristics of low bleeding and active expansion while setting are most important. During the grouting of a mine shaft in South Dakota it was repeatedly demonstrated that the small extra cost of the materials required to produce an expanding, non-bleeding grout was saved many times over in not having to go back and redrill and regrout the holes. The same is equally true of sanded cement grouts used to intrude coarse gravels, as in the preplaced aggregate method of making concrete. If expansion

[7] "Some Factors Influencing the Strength of Concrete Containing Admixtures of Powdered Aluminum," by C. A. Menzel, Journal, ACI, January, 1943.

is not provided along with low bleeding, much lower compressive strengths result.

Column 3 of Table 4 gives the Penetration Ratio, an approximate measure of the ease of grouting fine fissures. Thus, an increase of water-cement (w/c) ratio from 0.65 to 1.15 multiplies by 16/4 = 4 the number of sacks of solid materials that can be injected at a given pressure. This column is included to demonstrate how penetration and bleeding normally work against each other.

GROUTING MATERIALS AND THEIR LIMITATIONS

Portland Cement.—For sanded grouts and lean cement grouts required to fill large voids only, Type I cement is quite satisfactory, but for the grouting of sands, Type III cement with its much finer grind is highly desirable. Where this cannot be easily obtained Type II is sometimes chosen when its grind is finer than Type I and also for its improved sulfate resistance to underground waters.

Slag-cement.—Wet ground water-quenched blast furnace slag with a specific surface of 4,000 sq cm per g to 6,000 sq cm per g Blaine may either be used with sodium hydroxide as a reagent[8] or as a replacement for up to 70% of portland cement as in the Trief process.[9] This product exhibits a superior sulphate resistance and because of its fineness is often a better material for the grouting of medium sands. To date, the use of slag-cement of this kind has been limited to a few very large jobs in Europe where the cost of setting up the wet grinding mill could be justified. Portland blast furnace slag cement as manufactured in this country also recommends itself to grouting purposes on account of its fineness (average 4,700 sq cm per g Blaine) and excellent sulphate resistance.

Resin Gypsum Cements.—Certain resin-gypsum cements of proprietary manufacture are available[10] for extremely fast or controlled set. These are expensive for normal use, but have found extensive use in the oil well cementing industry for shutting off water in a hurry. They achieve an initial set in from 30 min to 90 min, and a compressive strength within 10 min or 15 min after initial set of approximately 1500 psi. A further advantage of this type of cement for special purposes is that it has a linear expansion of approximately 0.3% at time of initial set. This is a valuable advantage in remedial work because it gives the plug a tight bond with the walls of the casing or to the exposed surfaces of the formation. Full strength of approximately 4000 psi is attained 3 hr to 30 hr after initial set.

Clays and Bentonites.—Where the main purpose of grouting is to arrest water movement clay-cement grouts are undoubtedly the most suitable over a wide range, and their use has been adequately covered elsewhere.[11] In

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8 Technical Memorandum No. 6-419, "Influence of Manufactures Sands and Admixtures on Pumpability, and Evaluation of a Colcrete Mixture," "Influence of Chemicals and Mineral Fines on Pumpability," "Influence of Grading and Specific Gravity of Manufactured Sands on Pumpability," Corps of Engrs., Waterways Experiment Sta., Vicksburg, Miss.

9 "The Design and Construction of Avon Dam," by J. M. Bogel, R. M. Ross, and T. McMillan, Proceedings, I.C.E., January, 1959.

10 "Cal-Seal and Hydromite Catalog, 1959," Halliburton Oil Well Cementing Co.

11 "The Use of Clay in Pressure Grouting," by Glebe A. Kravitz, Proceedings, ASCE. Vol. 84, No. 1, February, 1958.

gravels some sand or silt may be included in the mix, whereas for sand careful choice of the clay and cement enable one to extend the usefulness of cement as the solidifying agent at least one extra sieve size, as shown in Fig. 4 and Table 1.

Locally occurring clays form a most economical and effective base for lean grouts. However, their chemical composition and physical characteristics vary so much from one location to another that each one should be individually investigated for suitability in the particular work. When dealing with clays, and especially bentonites, it is important to keep in mind the order in which the different ingredients are mixed. By mixing the bentonite first into the water, rather than the cement or putting them in together, it is possible to reduce the amount of bentonite necessary to produce a grout of a given consistency by about 45%. Again by predosing the mix water with a suitable electrolyte before addition of the bentonite it is also possible to effect further comparable savings in bentonite. Similarly, the addition of a fresh cement slurry to a prepared clay or bentonite slurry of essentially the same consistency frequently produces synergistic thickening to a marked degree.

Chemicals.—Pure chemical grouting is outside the scope of this paper, but combinations of clay and chemicals provide a useful range of materials for the grouting of fine sands that will accept clay. The most common chemicals used are sodium silicate with either sodium aluminate or hydrochloric acid as a reagent. The chrome lignins are also satisfactory jell-forming compounds, and are available either as crude sulfite liquor and chromic acid in liquid form or as a preblended powder that needs only to be mixed with water. Another combination consists of a mixture of two organic monomers which produce stiff gels from a dilute, aqueous solution by polymerization when properly catalyzed.[12,13,14] In all cases, jell time and strength can be controlled fairly accurately.

Examples of unconfined compressive strengths attainable in a medium sand with two of these clay-chemical grouts are given in Figs. 6 and 7. Fig. 6 shows the strength-silicate content ratio of a new single-shot silicate-based grout modified by the addition of small amounts of a synthetic ammonia derivative, and Fig. 7 the strength-concentration curve of a proprietary chrome lignin. Both of these grouts had clay contents on the order of 30% to 40% of the weight of the water.

Clay-chemical grouts also have a wide range of usefulness in medium sands for shutting off water, and in fact are at the lower limit of the suspended grouting materials that can be used for injection into sands.[15] When properly used, they are quite economical too, because the clay content reduces the requirements for active chemicals for a given volume of grout, and in practice have been found to give a better water cut-off than clay-cement grouts in the same materials.

The reason for this is that most formations are stratified or nonhomogeneous, some layers or zones being more permeable than others. A

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12 "AM-9 Chemical Grout and It's Application in Underground Mine Workings," by R. H. Karol. Presented at the February, 1959 Natl. Western Mining Conference at Denver, Colo.

13 "AM-9 Technical Data," Amer. Cynamid Co.

14 "Technical Memorandum on Terra Firma," Concrete Chemicals Co.

15 "French Grouting Practice," by Armand Mayer, Proceedings, ASCE, Vol. 84, No. 1, February, 1958.

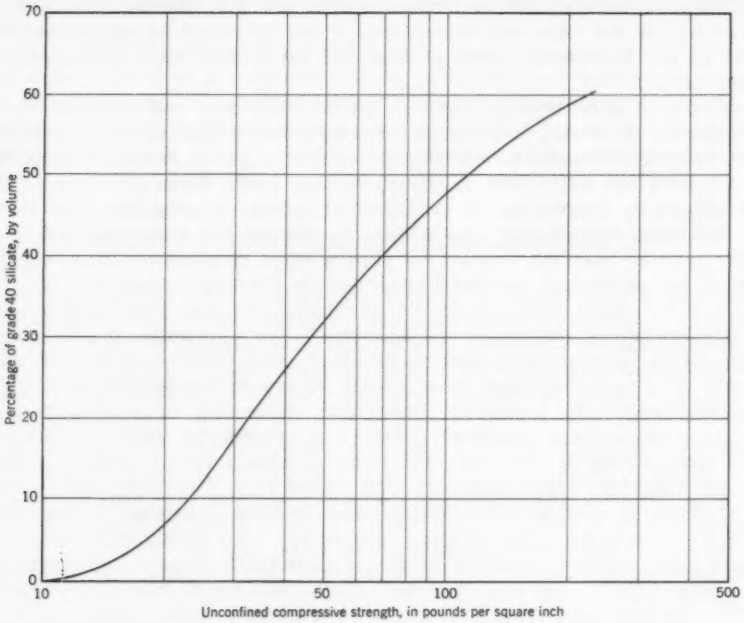

FIG. 6.—STRENGTH OF S.A.F. CLAY-CHEMICAL GROUT IN FINE SAND

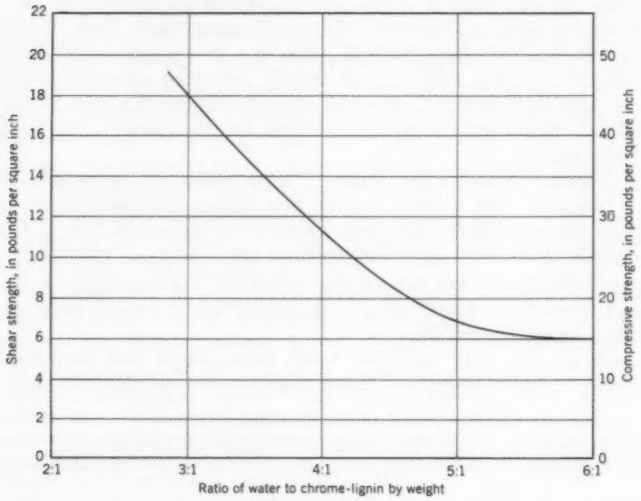

FIG. 7.—STRENGTH OF CHROME-LIGNIN CLAY-CHEMICAL GROUT

clay-chemical grout flowing through and filling the open-work layers will also penetrate the finer grained layers much better than cement or cement-clay grouts. Furthermore, even in the more permeable areas, the gelled chemical will completely fill the space even though the suspended material settles.

Pozzolans.—Fly ashes, opaline cherts, diatomites and calcined shales are frequently used for as much as 50% replacement[16,17,18,19] of cement. Under certain conditions in which high early strength is not required, hydrated lime may be combined with any of the above pozzolans to give good, economical grouts with cement replacement of up to 80%.[20] The addition of any of these lime-reactive materials, especially ground slag and calcined opaline shale considerably reduces the solubility of cement-based grouts. [8]

Sands.—Sand is a desirable material for use with portland cement to grout coarse materials such as large gravel, broken rock, and so on. For good pumpability with piston-type displacement pumps, minimum bleeding and minimum water requirement, the sand should be well graded from 95% to 100% passing U. S. Standard sieve No. 16, and from 10% to 30% passing sieve No. 100. A grout containing this sand (D_{95} = No. 16 sieve) will flow easily through in place material having a minimum (D_{10}) size of 3/4 in. and somewhat less readily into D_{10} = 1/2 in., the N ratios being 16 and 10, respectively. As the D_{10} size of material to be grouted becomes smaller, the D_{95} size of the sand should also be reduced.

Conversely, larger top-size sands may be used in larger void size materials, but as the maximum size of the sand increases, the ability of the valving systems of displacement pumps to maintain pressures decreases.

The proportion of sand to cement will depend on the type of results desired and the pumpability of the grout.[21] Strength and impermeability comparable to concrete may be obtained with one to two parts sand to one part cement, by weight, plus suitable admixtures to permit concrete-range water-cement ratios. At the other extreme, for the filling of space with the least expensive, rigid, non-erodable material, five to ten parts or more of properly graded sand have been used with one part of cement. As the mixes are made leaner, it is usually desirable to include suspending agents such as bentonite, clay, or certain chemical thickeners.

In connection with the grouting of coarse materials, it may be of value to examine briefly the stone and sand gradings widely used in two well known grouted concrete processes. For preplaced aggregate concrete by the Prepakt method in which the minimum (D_{10}) size of the coarse aggregate is normally limited to 3/4 in., it is interesting to observe that the standard specifications for the sand to be used in intrusion grout also calls for 95% passing the No. 16 sieve, giving an $N = D_{10}/D_{95}$ ratio of 16, indicating good groutability.[20]

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16 "Properties of Cements and Concretes Containing Fly Ash," by R. E. Davis, R. W. Carlson, J. W. Kelly, and H. E. Davis, Proceedings, ACI, June, 1937.

17 "The Use of Pozzolans in Concrete," by R. E. Davis, Journal, ACI, January, 1950.

18 "Fly Ash as a Pozzolan," by R. F. Blanks, Journal, ACI, 1950.

19 "What You Should Know About Pozzolans," by R. E. Davis, Engineering News Record, April, 1950.

20 "Guide Specifications for Grouting," by B. A. Lamberton, Intrusion-Prepakt, Inc., 1960.

21 "An Investigation of Sanded Grout Using Manufactured Sands, Miscellaneous Paper No. 6-195," Corps of Engrs., Waterways Experiment Sta., Vicksburg, Miss.

It should be noted that lack of smaller sizes in the coarse aggregate for use in grouted concrete as compared to natural gravels produces higher porosities on the order of 40%, with consequent higher pore diameter ratios. Reference to Fig. 1 shows that raising the voids to 45% (a not uncommon figure) gives a critical pore diameter ratio of 31%. The writers are aware of cases in which the coarse crushed rock for grouted concrete packed so openly due to its particular mode of fracture that the voids ran as high as 48%, that would raise the pore diameter ratio to 41%, or twice what could be counted on for a graded natural gravel.

Use is made of this opening up of the rock in the Colcrete method, in which a minimum size of coarse aggregate of 1-1/2 in. is usually specified, whereas the sand for grouting has essentially a D_{95} of 3/16 in., or No. 4 sieve, that would give an $N = D_{10}/D_{95}$ ratio of 8 before correction to take account of the higher void content of the rock. Based on a probable void

TABLE 5.—GRADING OF SAND IN PREPLACED AGGREGATE PROCESSES

U. S. Standard Square Mesh (1)	Cumulative Percent Passing	
	Prepakt (2)	Colcrete (3)
4	100	90 - 100
8	100	75 - 100
16	95 - 100	55 - 90
30	55 - 80	35 - 59
50	30 - 50	10 - 30
100	10 - 30	0 - 10
200	0 - 10	------
Range of F. M.	1.40 - 2.10	2.11 - 3.35

ratio[17] of 45% for the 1-1/2 in. plus stone, a corrected value for the effective N ratio might be N (31/20) = 12.4, an acceptable figure. Table No. 5 shows the standard specifications for grading of sand for use in both preplaced aggregate processes.[20,22]

Coarse Materials.—Where water is moving through the larger voids at sufficient velocity to wash away cement and sand, large particle suspensions such as pea gravel and aggregates up to 3/4 in. may be used to deliberately choke the passages and provide a filter that will catch and hold the finer materials. Other materials that have been used to plug large voids are sawdust, dung, chopped rope, hay, or cellophane, specially processed wood and bark chips, and ground walnut shells.[23,24]

Admixtures.—The term admixtures is used herein to cover materials used in relatively small quantities compared to the other ingredients of the grout

[22] "Grouted Concrete Construction," by S. Champion and L. Truman Davies, The Reinforced Concrete Review, June, 1958.

[23] "Silvacon and Silvacel Catalog, 1959," Weyerhaeuser Timber Co.

[24] "A New Report on Lost Circulation Materials," Halliburton Cementing Services.

for the purpose of modifying its properties both in the fluid and hardened state.

Surfactants, Retarders and Accelerators.—The use of calcium lignosulfonates and certain alkyl aryl sulfonates to reduce the water-cement ratio of high strength grouts is fully justified, because the lower water content will decrease the bleeding and improve the strength and watertightness.[25] For water-cement ratios up to 1.0 a lignin water reducing agent is desirable because it enables more fluid grout to be produced.

At higher water-cement ratios water reduction is not a factor, but dispersion is still available to aid suspension by preventing coagulation of the cement grains. Much more important, however, is that the grout should not bleed and that it should expand while setting. Lignins also act as retarders for the hydration of cement, that is advantageous in hot weather, and a necessity for the grouting of preplaced aggregate. In cold weather, accelerators, chiefly calcium chloride, are used to overcome this retardation when required.

Protective Colloids.—Protective colloids, thickeners, or chemical suspending agents, when used in very small amounts, are effective in reducing bleeding and segregation, and can even replace natural materials such as clay or bentonite to produce very lean and economical mixes. They have the advantage of being quite predictable in their performance compared to locally occurring clays, have no real grain size, and appear to give a lubricating effect to the grout that enables it to penetrate fine voids much better.

Gas Producing Agents.—Expansion of the grout, due to the formation throughout the mass of minute hydrogen gas bubbles generated by the reaction of the alkalies in the cement with finely divided aluminum powder, is now generally accepted as desirable. The presence of the many small bubbles gives a certain cohesiveness to the mix and helps reduce bleeding, while expansion during the plastic stage maintains contact and improves the bond of the grout with the walls and particularly the "ceilings" of the interstices into which it is forced. For the grouting of preplaced aggregate controlled expansion is absolutely essential in order to insure good bond with the undersides of the coarse aggregate, that is reflected in a considerable increase in strength[7] (as well as provide air-entrainment for durability where concrete made in this manner is exposed to the weather). This applies to the grouting of any coarse material in place.

A word of caution is appropriate. The quantity of aluminum powder required to produce the desired expansion is on the order of 0.01% by weight of the cement. This amounts to a few grams, a difficult quantity to weigh on the job. A much more reliable procedure is to premix the aluminum powder with a chemically inactive carrier material such as fly ash or pulverized limestone, the combination of which can be batched on the job by volume or weight without undue error. The choice of proper grade of aluminum powder and activator for correct time-expansion characteristics with various cements is also most important. A number of complete grouting fluidifiers are available that not only contain the correct grade of aluminum powder and activator, but also water-reducing and suspending agents.[26]

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[25] "Application of the Principle of Dispersion to Portland Cement," by E. W. Scripture, Research Paper No. 35, Master Builders Co.

[26] "The Use of Admixtures in Cement Grouts," by Alexander Klein and Milos Polivka, Proceedings, ASCE, Vol. 84, No. 1, February, 1958.

It is generally desirable to make sure that the expansion is three to four times as great as the bleeding. Delays, due to field conditions of mixing and pumping, frequently cut down on the attainable amount of the expansion of the grout in the work because of the large proportion of the total expansion that usually takes place during the first half hour or so, especially in warm weather.

Fine Solids.—Many of the fine solids are natural or artificial pozzolans, and as such can be used to replace part of the cement. Frequently they increase the resistance of the grout to sulfate waters. The addition of calcined opaline shale and ground, water-quenched, blast furnace slag has been shown to materially reduce the solubility of cement-based grouts.[8] Clays, silica flour, limestone dust and silica gels serve to produce a thixotropic mix with low bleeding, but add nothing to the strength of the hardened grout.

Lubricants or Primers.—The use of a lubricating chemical grout to prime the hole before the injection of a neat cement grout has been a fairly recent development and has proven most effective in the field for the grouting of fine sands and rock fissures. Under this system a batch or two of chrome-lignin or silicate-aluminate chemical grout is first pumped down the hole, followed by a high water-content cement grout. It has been found that this considerably reduces pumping pressures for the cement grout and assures much better "take." At times it has even been possible to get a hole to take cement grout by priming it with chrome-lignin grout after it had previously refused to take by any other means. It is particularly noticeable in grouting fine rock fissures that the chrome-lignin grout pumps at a lower pressure than plain water due to its excellent wetting properties and lower surface tension characteristics. There are also surface active agents available as primers that will reduce the surface tension to less than 30 dynes per cm, as compared to 73 dynes per cm for water.

METHODS OF GROUTING

The techniques of grouting in granular materials vary with the nature of the particular material and the intended purpose, and generally differ considerably from those used for rock foundation grouting. In rock a hole can usually be drilled and held open; in soils one must drive a pipe or casing, using drilling only to pass through boulders or previously grouted areas. In rock, a packer may be used to grout in selected zones anywhere along the length of the hole; in granular materials a packer is seldom used, and except when using the cuffed-tube the grouting is done in the zone at the bottom of the hole that is at the end of the driven grout pipe or casing.

Soft Fine Grained Saturated Soils.—Soils from which a portion of the water may be squeezed, but through which the flow of the liquid would be too slow for practical grouting with true chemical grouts, may be stabilized with cement and cement-sand grouts. The favored procedure is to drive a grout pipe, 3/4 in. to 1 in. diameter, to the lowest level to be stiffened, then slowly pump a bulb or layer of grout into the area. Considerable pressure may be required to start the flow, but once the grout moves, the pressure should be reduced to that which will not lift the ground. Here the authors agree that experience is still of more value than theory, at least until the engineer learns that a high-pressure start followed by good take at low pressure means a lateral sheet-type of flow that will raise the area if the pressure is greater than the

static weight of the ground above the point of application. This will squeeze a little water from a relatively large flat area. On the other hand, continued high pressure with low take is a general indication of a bulb-type displacement that will force more water from a limited volume of soil. When the engineer decides that the grout has spread far enough in the first instance, or that the pressure or quantity is sufficient in the second, the pipe is raised a foot or so and the process repeated until the ground cover is insufficient to permit enough pressure to make the operation worthwhile. Grout is also usually pumped while the pipe is being raised to provide "pencils" of solid materials that probably add to the stability of the grouted area.

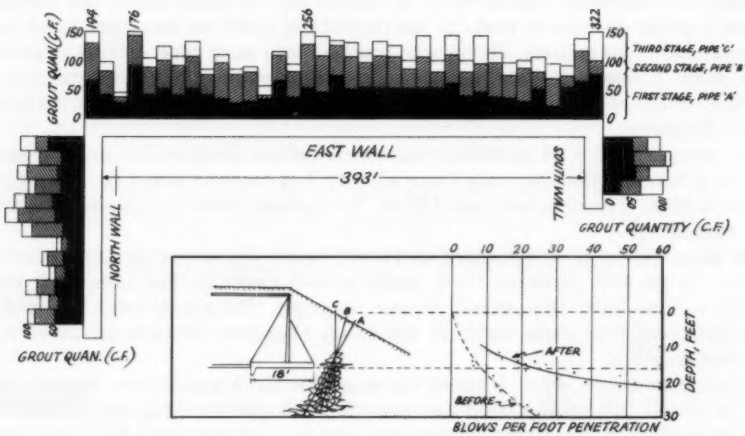

FIG. 8.—COMPARISON OF TYPICAL BLOW-COUNT BEFORE AND AFTER GROUTING

Grouting is then shifted to the next pipe that may have been driven in advance or not until needed. Pipe spacing will be from 5 ft to 10 ft for the layer type of take, and from 2 ft to 5 ft for the bulb type.

The job may be considered successful if the blow count is at least doubled for check pipes driven after the grout has hardened as compared with the count for the first few pipes driven. Therefore, the engineer should record initial blow counts at several levels, as well as near the finish.

The stabilization of the foundation material under three walls of the Snelling Avenue Reservoir, St. Paul, Minnesota, is believed to be a good example of this technique. Referring to Fig. 8, the contractor first grouted through the row of vertical pipes marked "C." He then grouted row "B," and finally at higher pressures, row "A." Quantities of grout pumped are shown in the bar graph. A typical blow count "before and after" record is also shown.

Sandy Materials: Stabilizing a Compressor Foundation.—In the example of a method for stabilizing medium sand with cement grout that combined consolidation and penetration the owners and engineers decided that stabilization of foundation material under a 2,000-ton compressor foundation block would reduce visible vibration that was rupturing pipe connections. The effectiveness of the grouting is attested by the fact that vibration was reduced by 90%.

Although before and after samples were not taken that might have shown the extent of grout penetration versus consolidation, a qualitative description of the foundation material is as follows in descending order: 3-1/2 ft of yellow sandy clay, 10 ft of yellow medium sand, 41 ft of yellow medium sand with up to 25% pea gravel and 10% coarse gravel, then sandstone.

Initial operations began with the drilling of holes 9 ft deep on 10-ft centers through the concrete floor surrounding the foundation block in a line 3 ft from it. This first pumping was intended to tighten the material under the slab and provide a grout curtain to restrict the flow of the grout in the upper zone during subsequent operations. Grout was pumped into each pipe until it appeared at the point between the floor and the vibrating compressor. Approximately 100 cu ft of grout was pumped in this operation. The floor slab was raised in several locations.

The second phase of grouting was started after drilling and driving holes 10 ft deep through the concrete floor slab on 3 ft centers and 6 in. away from the foundation. These holes took 110 cu ft of grout. Slabs and platforms were again raised.

For phase three it was decided to return to the phase one or grout-curtain line, but drive the pipes to 15-ft depth at 6-ft centers. The lower 6 ft were medium yellow sand that would accept no grout. The pipes were then pulled a few feet until the ends were in the sandy clay into which some 50 cu ft of grout was pumped.

Phase four holes were located close to the block and driven to a depth of 21 ft to reach the yellow sand and gravel. Pressure washing was tried in an effort to remove some of the fine material and thereby increase the grout take. The washing effect was negligible, however, as the quantity of grout pumped amounted to only 30 cu ft.

At this point, advantage was taken of a slow compressor speed period to return to the phase-one area in which 130 cu ft more of grout was pumped. Because of the reduced vibration, higher pressures could be used without leakage along the grout pipes to the surface.

Finally with the compressor shut down, angle holes were drilled through the foundation block to intercept the soil-concrete contact. The grout take was 33 cu ft, indicating a considerable void under the central portion of the block.

After the compressor had been put back into operation 18 hr later, vibration readings were taken that showed that the magnitude had been reduced approximately 90% from that indicated by readings taken before stabilization was undertaken.

The grout used consisted of two parts portland cement to one part fly ash plus 2% calcium chloride by weight of the cement, water and a small amount of a water-reducing, set-retarding admixture that also provided for expansion of the fluid grout.

Settlement of Buildings under Construction Halted by Combination Grouting.—A good example of grouting using both cement- and chemical-based

grouts with before and after load tests was provided during the construction of a large industrial building in up-state New York at which serious trouble with foundation settlement was quickly and effectively halted by the use of a combination of these two types of grout.

Initially, the footings for the steel columns of the building had been designed for a soil bearing value of 5 tons per sq ft, but actual excavation revealed inadequate values at the proposed foundation levels. In certain areas trench excavation was carried down 15 ft to 20 ft under the footings in an attempt to reach satisfactory bearing (Fig. 9). Subsequently, these excavations were backfilled with granular material, mechanically tamped to 95% compaction. The backfilling and tamping had to be carried out in extremely cold weather (-20°F), and although the excavations were covered with tarpaulins and heated with salamanders, much of the backfill was probably frozen when placed.

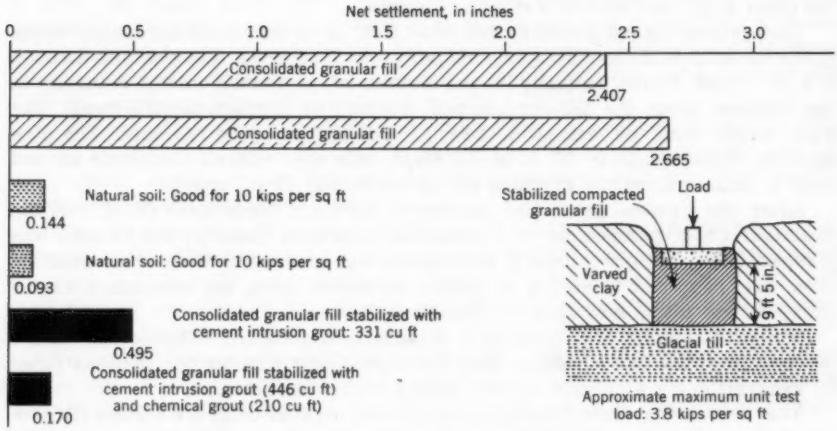

FIG. 9.—FOUNDATION LOAD TESTS EFFECT OF GROUTING ON SETTLEMENT

After the concrete foundations were placed and the structural steel erected, it was found that many of the footings, especially those on the granular fill, had settled as much as 7 in. Because the output of this plant was vital to the owner's new product line, it was essential that corrective measures be quickly carried out so that remaining construction work could be finished and the plant put into operation.

In addition to test borings made through the fill to check the compaction, four full scale load tests were carried out on column footings supported on both the fill and the natural soil, a glacial till. The footings on natural soil settled about 1/4 in. whereas those on fill averaged 2-1/2 in. of settlement.

Two methods of correcting these conditions were considered. The first was to underpin the foundations with piles, driving them to the firm underlying glacial till. However, considerable difficulty could be expected in driving such piles because head room was restricted by the already erected struc-

tural roof members. Accordingly, the second method, stabilization grouting was selected.

Of the first two footings, one used both cement-based grout and chemical grout whereas the other used cement-based grout only. The chemical grout used in this case was the two-shot silicate-chloride process. Under full test load, the footing on the combined grouts settled less than 3/16 in., and the one on material stabilized with cement grout a little less than 1/2 in. (Fig. 9).

These results were considered so favorable that the owner instructed the contractors to go ahead with grouting under all the footings on fill that had shown settlement. This is a particularly useful example of grouting because it shows what can be achieved by the combination of both cement and chemical grouts used together to achieve desired results by the most economical means.

The cement-based grout consisted of two sacks of portland cement (Type I), 1 cu ft fly ash, 2-5/8 lb admixture and 15 gal to 20 gal of water. The chemical grout was made up of two solutions, one a 50% solution of sodium silicate and the other a 30% solution of calcium chloride.

The cement based grouting was done first in order to fill the larger voids in the backfill with the stronger, cheaper grout. The grout was pumped through 3/4 in. pipes driven initially to the bottom of the trench until it returned to the surface along the fill-trench wall contact or through the fill itself. The pipe would then be raised a foot or two and pumping resumed. The pipe spacing, approximately 10 ft at the start, was split spaced each time around until in many places the grouting points were only 15 in. apart.

After the cement grout had hardened, the finer voids were filled with the chemical grout pumped as two separate solutions. Usually, the silicate was pumped first, but occasionally when there was an excess of chloride solution over that required to satisfy a silicate saturated area, the chloride would be pumped first into a new area to dispose of the batch.

Cement-Chemical Stabilization to Facilitate Excavation.—The effectiveness of combining the use of cement and chemical grout was further demonstrated by excavation for a culvert in stabilized granular material.

The contractor first tried to jack a 54-in. pipe through a railroad fill, but found the material too resistant. He then tried excavating and placing sectional corrugated steel plate, but was again stopped by excessive flow of material into his excavation that caused the fill to settle.

In this situation, the grouting contractor drove horizontal pipes most of the way through the fill from a shored heading, then pumped a cement-flyash grout with an admixture to fill the voids in zones of fine to medium gravel, the pipes being withdrawn a few feet at a time as the grout was pumped. He then filled most of the voids in the finer grained material with a single-shot silicate grout. This procedure followed a test program during which a number of chemical grouts were tried, the results leading the contractor to choose the combination he actually used.

Excavation following stabilization was relatively simple, the silicate grout being stable but not very strong and the cement grout relatively hard, but in limited sized areas easily broken or pried out.

Cement-Clay Grouting in Mixed Materials.—Soil boring tests made along the line of the Dartford Tunnel under the Thames River in London, England showed that on the north bank the tunnel would have to be driven through an old ash fill containing bricks and other debris. Because the tunnel was to be

driven under pressure, it was feared that this porous zone would allow the compressed air to escape and endanger the driving of the tunnel. To insure against such a contingency, it was decided to grout this old ash fill with a clay-cement grout for a length of 90 ft south of a concrete tank-type cofferdam that would eventually house the ventilating plant.

The contractor first drove two lines of steel sheet piles outside the area of the tunnel 46 ft apart and some 90 ft long to contain the area of grouting. One-inch diameter extra heavy grout pipes in 5-ft lengths were then driven by hand-held pneumatic hammers on 8-ft centers to depths varying from 20 ft to 25 ft; a pattern of seventy pipes in all. The perimeter holes were grouted first using a mix of 112 lb of portland cement, 56 lb bentonite and enough water to make 5-1/2 cu ft of grout. The grout returns occurred mostly on the inner faces of the sheet pilings. The pipes were withdrawn 2 ft at a time until approximately 15 ft of pipe was still left in the ground, when the grout usually returned along the pipe making it impractical to pump any further on the hole. The pipe was then withdrawn completely.

For all inner area pipes, a mix of 112 lb of portland cement, 112 lb of bentonite and enough water to yield 11 cu ft of grout was used. The grout was very fluid, and gave approximately the same flow cone reading as plain water in the standard flow cone consistency apparatus[25] but, nevertheless, the bleeding shrinkage after three days was only 1.1%.

Although 465 cu yd of grout were pumped into an intended volume of 1000 cu yd of ash and broken brick, a definite statement of void content is not warranted, inasmuch as unknown quantities escaped into the pilot heading driven below and perhaps elsewhere.

During excavation after stabilization, it was found that the grout tended to remain in the vicinity of each pipe, except where there were fissures along which it had run for some distance. There were also places where the grout had formed large bulbs that had squeezed and thereby consolidated the surrounding material.

Tunneling through the stabilized area was relatively simple. The tunnel muck was dry and easily removed, whereas the remaining material required only light support and permitted the loss of very little air. The grouting operation was, therefore, considered completely successful.

Grouting with Sand and Cement in Coarse Granular Material.—The feasibility and method of consolidating coarse material with a low water content sand-cement grout will be illustrated by work accomplished at Ashton Dam.

The west end of this dam, between the power house and abutment, consisted of upstream and downstream retaining walls with rockfill between surfaced with gravel and cinders. It was noted that the backfill was subsiding. Closer observations revealed that cracks were opening in the downstream wall and that it was bowing. It was imperative that the load on the wall be reduced or removed, or the wall reinforced. After studying the problem the owners decided to relieve the load on the wall by grouting the backfill and to strengthen the wall by anchoring it to the stabilized backfill.

The walls are 33 ft apart and 70 ft high as they leave the power house and 74 ft apart at their contact with the abutment approximately 70 ft away.

The general method of grouting was to drill holes from the surface and insert 1 in. pipes to full depth, then after placing the desired quantity of grout through a given insert, to raise the pipe a few feet. This procedure was repeated on all the holes until the grout surface reached the top of the backfill.

A series of weep holes were drilled in the downstream wall to prevent build-up of hydrostatic pressure as well as to give grout level indications. These holes were carried well into the rockfill so that when grout appeared, a 7/8 in. deformed bar could be inserted to anchor the wall to the grouted backfill. Seepage through this section of the dam, estimated at 200 gpm before grouting, was completely stopped.

The mix used for the grouting was normally 1 sack of cement, 1 sack of calcined shale pozzolan, 4 cu ft of sand, 2 lb of admixture and 11 gal to 12 gal of water.

The contractor was on the job approximately 40 days. During this time, he drilled 1,800 linear ft of holes and placed 36,000 cu ft of grout.

Drilling, Washing and Grouting.—When the drilling-washing-grouting procedure is applied to granular materials, it is usually for the purpose of placing a grout curtain in alluvial deposits ranging in size from fine sand to large rocks and boulders. The method is expensive in terms of labor, equipment, and time and is used as a last resort short of excavating and replacement with a tight material.

The procedure consists of, (1) drilling through the zone previously grouted, (2) jetting fines from the next zone below, and (3) grouting the remaining relatively clean gravel with a low water-content grout. When the grout has hardened, the procedure is repeated. The work usually starts at the surface, the first drilling being through a concrete slab placed for the dual purpose of forming a working area and a pressure lid. By proceeding from the top downward, the drilling is always through hardened grouted aggregate. This not only provides a stable hole for the continuation of the work, but also yields good information on the quality of the grouting work completed.

To the best of the writers' knowledge, the first real test of this procedure was carried out[27] for the Corps of Engineers at the Folsom Dam Project in 1950 to determine the feasibility of establishing a cofferdam cut-off in riverbed material too rough to permit the use of steel sheet piling.

The site of the work was a sand-gravel bar on the American River in the vicinity of the dam site. After placing a concrete working pad 6 ft to 8 ft wide, short sleeves of 4-in. pipe casing were set in this slab in two rows 15 in. apart, with the holes staggered at 30-in. spacing. Jet pipes carrying air and water at 100 psi alternately and together were inserted in each sleeve. The return flow (Fig. 10) brought up most of the fine materials (below 1/4 in.) from a bulb-like zone 3 ft to 5 ft in diameter. The return was through 2D, 3D, 4D, 5D, 2U, 3U, 4U, and 5U. The average depth of the holes was 14.4 ft in row D and 11.3 ft in row U. Beyond this zone depth, the area through which the return water flowed reduced the velocity to the point at which material loosened at the jet tip was no longer carried to the surface.

The water and air were then shut off, a grout connection made, and the voids in the aggregate pocket filled with a grout composed of cement, fly ash, fluidifying admixture, calcium chloride and water. Fine sand was also used in somewhat more than half the mixes to control the flow of grout through open strata, while the calcium chloride was used to speed up the hardening and make drilling for the next stage possible with minimum delay. By these means a grout curtain approximately 70 ft long and 20 ft to 30 ft deep was

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27 "Experimental Grouting of River Gravels, Folsom Project, American River," Sacramento Dist., Corps of Engrs., August, 1951. Supplement No. 1, February, 1952.

FIG. 10.—JETTING GROUT HOLE NO. 3D WITH AIR

FIG. 11.—UPSTREAM SIDE OF TEST SECTION

established with the pumping of a total of some 5,200 cu ft of grout. Fig. 11 shows the upstream side of test section. Note the lateral flow of grout in the segregated pervious strata below well graded sand and gravel embankment built up by the bulldozer.

During the following year, the test section was subject to two floods of record as well as the normal seasonal high water period. When examined during a low water period, it was found that much of the sand and gravel adjacent to the section had been washed away to expose the grouted cut-off. The upper portion of the wall is in material above water table built up by a bulldozer that mixed the aggregate. Here the wall consists of cylindrical, bulbous columns that vary in diameter from 12 in. to 30 in. The lower portion of the exposed wall was grouted in undisturbed gravels and shows that the grout flowed laterally in the more open strata.

It is reported that check holes drilled between the grout pipes at the time of construction showed considerable water loss in many instances. Considering the bulbous nature of the curtain, it is believed that most of this leakage was to one side or the other rather than to both sides at once that would indicate a hole in the wall. On the other hand, no claim should be made that this wall was impermeable, although drilling, pressure testing, and grouting another line of holes down the center would all but guarantee watertightness. It should also be remembered that if there are leaks, the wall itself is of low to intermediate grade concrete and cannot be washed out by piping.

That a cut-off of this type can be made tight was proven at Doshi Dam in Japan where a 28-ft high radial concrete cofferdam 180 ft long (Fig. 12) was constructed over a 21-ft deep drilled, washed and grouted cut-off placed in a riverbed containing all sizes of material from fine sand to 10-ft boulders. A section of the cofferdam and grouted cut-off are shown in the insert of Fig. 12. Almost as soon as completed, the cofferdam was overtopped by 3 ft of water that, falling on the downstream side, removed riverbed material to a depth of about 10 ft exposing but not damaging the cut-off. During the following construction season, the work area for the main dam was pumped out and excavated revealing less than 10 gal per hr leakage through the cofferdam cut-off.

Another very interesting and useful application of the drill-wash-grout method was used at the Tappan Zee Bridge that carries the New York Thruway over the Hudson River. The four main spans of this structure are supported by hollow concrete box caissons that support a portion of the bridge load by their buoyancy and transmit the rest to bedrock through 30-in. concrete filled pipe piles.

During driving, many of the pipe piles reached "refusal" at depths of 260 ft to 280 ft, some 20 ft short of bedrock, at which they hit large boulders, apparently part of an ancient till deposit containing boulder clay with scattered sand pockets. When it was found impractical to drive the piles to bedrock, it was decided, in effect, to bring the rock up to the pile by replacing the fines from around the boulders with a strong sand-cement grout (Fig. 13).

The procedure used to do this was approximately as follows: First, the pile was cleaned out, then after placing a 5-in. pipe casing in the pile center, the pile was filled with clean, coarse aggregate that was grouted to produce preplaced aggregate concrete. A 4-in. diamond bit was next used to drill the plug from the end of the casing and through the boulder that had stalled the pile.

The drill was then withdrawn and replaced with a pipe having a specially designed tip that directed four jets of water upward at an angle of 45° and one straight down. Jetting pressure started at 50 psi and increased gradually to about 200 psi. The jetting action loosened and flushed out mud, sand, and gravel up to 2-in. diameter. When jetting in till, the washing was continued until the exit water was clean. In the sand lenses, the in-place material was less stable so that the effluent would not clear. In such instances jetting was stopped when the amount of material removed was judged sufficient. The depth-of-washing stage was 4 ft to 5 ft in clay-till materials and on the order of 3 ft in the sand lenses.

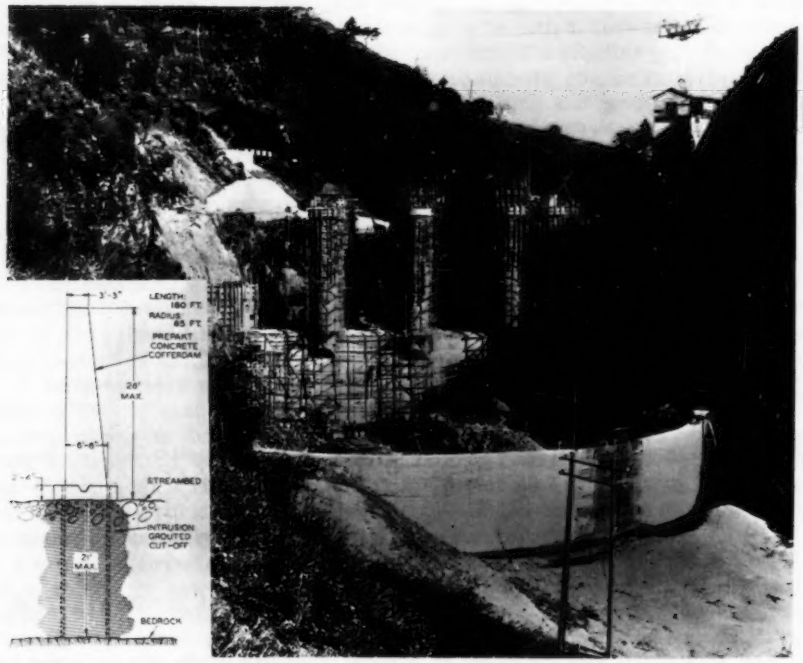

FIG. 12.—DOSHI DAM; CONCRETE COFFERDAM AT UPSTREAM END OF SITE

On the completion of jetting at any stage, a thick grout was pumped into the area until the pressure reached 200 psi—the estimated skin frictional resistance of the pile.

To prevent waste of grout in the annular space between the grout pipe and 5-in. casing and to minimize the likelihood of the two being concreted together, it was suggested that a gasket be used at the top of the casing to prevent escape of water during grouting. As long as this water could not escape, it acted as a very effective packer and prevented grout rise in the annular space. Nai Yang, F. ASCE, resident engineer on the project, with the cooperation of the sub-contractor's job engineer and superintendent, developed a

very effective pressure sensing system around this gasket by adding a pressure gage to the casing (Fig. 14). Before the upper area of the washed pocket was covered with grout, the grout-displaced water could escape with relative ease through the soil, but as grout filled the bulb-like area, it sealed off the pores so that the pressure rose in the second gage. Grout pumping was continued until the gage pressures on both the grout supply line and the casing were equal. At about this point, the casing was allowed to bleed a little to permit diluted grout to move out of the pocket and up the annular space. The pressure was then held constant for an hour after which the grout pipe was removed. Diluted and excess grout was washed out to a depth of 10 ft to 15 ft above the tip of the pile to complete the stage of grouting.

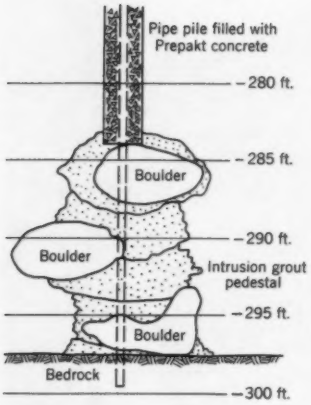

FIG. 13.—GROUT PEDESTAL BUILT FOR TAPPAN ZEE BRIDGE

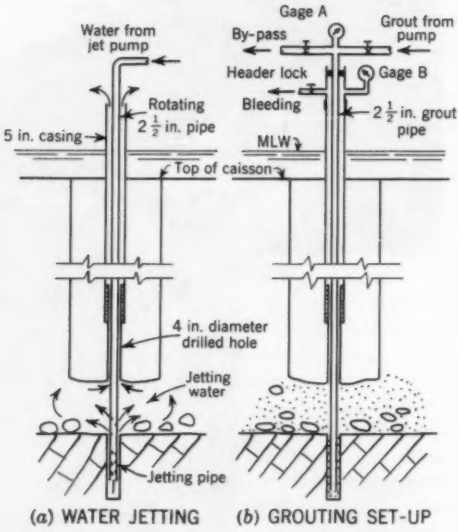

FIG. 14.—PRESSURE SENSING SYSTEM

The procedure was repeated in 5 days. Use was made of 4-in. steel bits to drill out the grout at the lower end of the casing and Nx diamond bits to core the zone of the preceding stage.

The grout mix consisted of 5 sacks of cement, 1 sack of fly ash, 6 cu ft of sand, 2-5/8 lb of admixture, and 32 gal of water. Compressive strengths taken on sections of cored grout removed at 5 days to 7 days and cured in the laboratory averaged over 4,000 psi at 28 days and over 5,500 psi at 90 days.

Grouting in Strata of Varying Grain Sizes.—Where maximum watertightness in stratified material of variable grain sizes is desired, as for a cut-off under a dam, it is necessary to vary the composition and consistency of the grout to suit the characteristics of the individual strata. Layers of coarser materials may accept sand-cement or cement-flyash grouts, whereas other strata may be penetrated only by very fine grained suspensions or chemical solutions. Several methods are available to accomplish this. One way is to

stage grout each layer as the grout pipe reaches it after first carrying out a water pressure test to confirm the nature of the in-place material. The drill-wash-grout method previously mentioned illustrates the procedure when applied to coarse gravels and rocks. In finer materials washing is ineffectual and penetration is obtained by choice of grout composition. This method has the advantage of relative simplicity, but it tends to be time consuming because each grouted zone must be allowed to harden before grouting the next lower stage.

A second method is to drive or drill a pipe to the bottom of the desired curtain, then grout at various levels as the pipe is withdrawn. This is the procedure most often used for stabilization by squeezing rather than penetrating the in-place materials. The method appears feasible for use in moderate sized gravel that will permit a casing to be jetted, fish-tailed or otherwise driven to position, followed by the insertion of a smaller pipe to be used for flushing out fines and grouting of the remaining coarse materials. This method has the advantage of separating the drilling and grouting operations, and permits repetition of the washing and grouting cycle as soon as the previously placed grout is hard enough to resist washing.

A third method especially suitable for grouting to great depths and in the presence of large boulders, involves the use of the "cuffed tube," developed in France.[12] After mud drilling the hole to full depth, a perforated pipe or cuffed tube is lowered into the hole. The perforations in this pipe are usually at 1 ft centers, with each hole being covered on the outside by a rubber sleeve that allows the grout to flow out under pressure. The mud is then washed out of the pipe and the drill hole, and the annular space filled with a weak cement-sand grout.

A double-sleeve packer is used to straddle a hole in the cuffed tube at any desired elevation, so that when grout pressure is applied it expands the rubber sleeve, ruptures the weak mortar around it and flows out to penetrate the soil. Although the installation of cuffed tubes is expensive, it appears to be one of the most satisfactory methods for grouting at great depth in granular materials. It provides not only selectivity of the zone to be grouted, but if necessary, the ability to return to any zone for regrouting.

Lubricated Grouting.—Lubricated grouting is a term proposed by the writers for a relatively new technique for facilitating solid suspension grouting. The method briefly consists of priming the hole with a dispersing agent before pumping the grout and the use of a chemical lubricant to suspend a dilute cement grout that does not bleed.

When the void size of the granular material being grouted is in the critical range for the size of suspended particles in the grout, the suspended particles frequently build up a filter cake around the end of the grout pipe that effectively prevents further flow of grout. If, however, a quantity of certain chemicals is pumped first, the grout particles seem to pass more easily through the zone of rapid pressure drop around the end of the grout pipe. Chemical solutions that have acted as primers are some of the silicates, concentrated lignins and silicate- and lignin-based chemical grouts.

Several theories have been advanced to account for the beneficial action of primers. Some believe that the primer acts as a lubricant. Others go a step further by suggesting that reduced surface tension encourages the grains to pass through finer voids. Still another theory is that the action of a primer is that of a dispersant, preventing agglomeration of grouting particles with

the grains of sand around which they are passing. Primers have made cement grouting possible in holes that refused plain cement grout.

A good example of the use and effectiveness of lubricated grouting was provided by the stabilization of part of a Polaris launch pad at the Cape Canaveral Missile Testing Center in Florida. Each hole was primed with a batch of a proprietary chrome-lignin chemical grout (formula and method of use are subjects of pending patent applications) that materially reduced the initial pumping pressure and facilitated the penetration of the lubricated cement grout that followed. The soil was a fine, slightly shelly, silty sand whose D 10 size was definitely below the normal limit for cement based grouts, as can be seen in Fig. 15. In order to test the effectiveness of the chrome-lignin primer and the chemical suspending agent, attempts were made to inject a plain cement-water grout of the same mix (30 gal per sack) without the suspending agent and without first priming the hole, but these met with complete

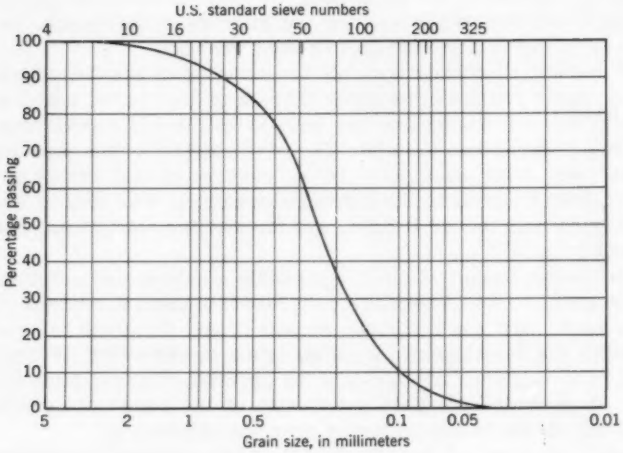

FIG. 15.—CAPE CANAVERAL SOIL GRADING CURVE

failure. The combined use of both primer and suspending agent enabled some 4000 cu ft of grout to be injected into the low density soil areas.

CONCLUSION

It seems likely that there will always remain a large amount of "art" as opposed to "science" involved in the process of grouting, but this paper has attempted to set out and clarify some of the basic principles of this "art" both as regards the design criteria for the materials involved and for the methods and techniques to be used to insure success.

1. It has been shown that the effective pore size of a grouted material is approximately one fifth of the grain size, and from this is derived the criterion for groutability, $N = D_{15}/D_{85}$ of 15.

2. When the ratio $N = D_{15}/D_{85}$ approaches 8, the criterion for groutability should be re-examined on the basis of $N = D_{10}/D_{95}$ in order to be doubly sure that the proposed grouting material is still suitable.

3. The application of this groutability ratio has been related to the Prepakt and Colcrete methods of producing grouted aggregate concrete.

4. The criterion for groutability developed in this paper is designed for reasonably homogeneous or dense grouts in which there is likely to occur particle interference at the critical entrance passageways of the grouted material. Extremely dilute grouts on the order of 500 gal or more per sack of cement such as are sometimes used in rock grouting for mines, are outside the scope of this paper.

5. The roles of various grouting materials and their limitations have been examined in the light of the previously derived criteria for groutability.

6. The importance of bleeding has been stressed and the solution to this problem tied in to various admixtures and combinations of grouting materials.

7. The use of certain chemicals or chemical grouts to precondition holes in order to insure a better "take" for cement or clay grouts is an innovation which has proved very successful in certain applications.

8. Recent developments of certain chemical and clay-chemical grouts have been mentioned in outline but no attempt has been made to cover all those available because many of them are proprietary and their composition closely guarded secrets.

9. Methods of grouting are covered fairly broadly and in some detail with examples taken from various successful grouting jobs in various parts of the world. The examples show how each situation is examined and treated according to the grouting method most suitable for it.

Although much more scientific research is needed, the paper provides the basic information necessary to examine a grouting problem in the light of generally accepted criteria and establish a range of grouting materials and an outline of procedure which will give a very good chance of success.

ACKNOWLEDGMENTS

The authors wish to express appreciation to James M. Polatty, F. ASCE, of the Concrete Research Division, Corps of Engineers, for his encouragement as well as assistance in advising on sources of information; to Raymond E. Davis, F. ASCE, Consulting Engineer, for advice and assistance; to the Waterways Experiment Station, Corps of Engineers for the loan of several technical publications; and to the Sacramento District, Corps of Engineers, for comment and permission to use the official photographs of the drill-wash-grout procedure at the Folsom project, Figs. 10 and 11.

Techniques developed by contractors are not often released for publication. The authors are therefore especially grateful to the following companies for permission to disclose data and job information and for the use of illustrations as indicated: Intrusion-Prepakt, Inc., Cleveland 14, Ohio (Figs. 8, 9, 13, 14, and 15); Lee Turzillo Contracting Co., Brecksville, Ohio; Edmund Nuttal Sons, England; Nishimatsu Construction Co., Japan (Fig. 12); Concrete Chemicals Co., Cleveland, Ohio (Figs. 6 and 7).

APPENDIX. ADDITIONAL REFERENCES

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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

SYMPOSIUM ON GROUTING: GROUTING OF FORT CAMPBELL THEATRE BUILDING

By Bruce E. Clark[1] Aff. M. ASCE

FOREWORD

This paper is one of several papers that constitutes the Symposium on Grouting that occupied two sessions of the June, 1960 meeting of ASCE in Reno, Nevada. Credit for organizing this symposium program goes to a Special Grouting Symposium Committee consisting of Stanley J. Johnson, F. ASCE Chariman, T. William Lambe, F. ASCE and W. K. Seaman, F. ASCE.

It is believed that these several papers constitute a significant contribution to engineering practice in the art and science of stabilizing foundation materials by grouting. This is a field in which there appears to be increased interest among foundation engineers and one which is expected to become more important as time goes on and the need to build structures on difficult soil conditions increases.

The papers of the symposium are concerned with cement, clay, and chemical grouting of seams, cavities, and granular materials. The papers illustrate the increased use of chemicals for grouting purposes and also their use with more conventional grouting materials such as clay and cement; grouting of overburdened materials that is rapidly becoming a more definite art and science and that, to some degree at least, is susceptible to laboratory studies; the grouting of a cut-off in granular materials to prevent underseepage beneath a major hydroelectric project (perhaps the first application of this type of grouting in the United States); and the grouting of subsurface voids and solution channels in limestone areas subject to large subsidence.

Note.—Discussion open until September 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings, of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

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Organized in 1952 the Grouting Committee with Task Committees on Cement, Clay, and Chemical Grouting has sponsored groups of papers or symposia occupying the sessions of meetings of the Society in 1954 and 1956, as well as 1960. Papers on grouting that have been published in the Journal of the Soil Mechanics and Foundations Division are as follows: Numbers 1145, 1204, and 1544 to 1552 inclusive. The report of the Task Committee on Chemical Grouting appears as Paper 1426 in the November 1957 Journal. A draft of a report on clay grouting has been prepared by the Task Committee on Clay Grouting; and the report on cement grouting prepared by the Task Committee on Cement Grouting is substantially in its final form.

The Administrative Committee on Grouting takes this opportunity of expressing its appreciation to the task committees for the excellent manner in which they have carried out their assignments and to the individuals who, as authors of papers, have made significant contributions covering the practices of grouting in various fields.

Raymond E. Davis, F. ASCE, Chmn.,
Administrative Committee on Grouting
Prof. of Civ. Engrg., Emeritus, Univ.
of California, Berkeley, Calif.

SYNOPSIS

The occurrence of a sink 12 ft in diameter and 3 ft deep inside a concrete framed theatre building at Fort Campbell, Kentucky is described. Exploration before and after its occurrence indicated that this sink was not directly due to collapse of an underground cave in the underlying limestone rock but rather to the collapse of a void in a weak, pervious zone in the lower part of the overburden. This zone has been found in approximately half the holes drilled on the base. To protect the building against structural damage resulting from further foundation failure, the overburden under the sink was grouted with a cement and cement-sand grout. Mixes designed and tested with two different locally available sands clearly showed that a well graded sand is more economical and yields a stronger product than a poorly graded sand with an excess of fines. A total of 34 cu yd of grout were pumped into holes drilled in the sink. Although subsequent exploration was unable to find any grout outside the immediate area grouted, it strongly suggested that grouting had tightened up the overburden under the entire building. The building was completed without further incident.

INTRODUCTION

The fact that limestone can be a treacherous foundation is quite well known. The number of structures on limestone that have encountered serious founda-

tion problems could hardly be determined, but there have been some notable ones such as Madden Dam and Hales Bar Dam. The principal problems related to limestone foundations arise from the fact that the top of rock line can be most irregular, and that there may be numerous and sometimes very large caves and solution channels in the rock.

The foundation trouble at the Fort Campbell theatre was quite different from this. The bedrock at Fort Campbell is limestone shot with caves and solution channels, but the theatre's foundation trouble was in the overburden, that is a thick residual clay. Into that clay was pumped 34 cu yd of a cement and cement-sand grout. Although this is a small grouting job it illustrates a type of foundation condition that is quite frequent in which there is thick residual clay on limestone. This condition is common in the central and southern United States. It is, therefore, most likely that builders have been or will be confronted with similar conditions elsewhere.

DESCRIPTION OF SITE

Fort Campbell straddles the Kentucky-Tennessee line, 50 miles northwest of Nashville, Tenn. The base was built during World War II, with no particular concern given to its location in the middle of a region of limestone sinks and caves. There are hundreds of large caves in the region. Typical are the Dunbar and Mammoth caves, that are 8 miles and 85 miles distant, respectively. There are millions of sinks in the region; sinks are depressions caused by the collapse of caves, and related underground erosion. The rock under Campbell contain an intricate maze of cave-like openings. This situation caused the base security officers some concern in 1956, when a hole 50 ft deep suddenly appeared behind a service station near the base, exposing a small cave. Speleologists (cave scientists) who were consulted assured the base authorities that most of the underground openings in the area were small.

The buildings constructed at the site during World War II were wood frame structures set on shallow pier foundations without grading. None, so far as the record shows, had any foundation troubles. However, much of the area had no surface drainage. Run-off water simply flowed into the sinks that drained into the underground cave system. After heavy rains, large areas surrounding the sinks were flooded. It was discovered, subsequently, that a gravelly pervious zone from approximately 0 ft to 20 ft thick occurred at the bottom of the overburden that is usually from 30 ft to 50 ft thick. Ninety-five wells were drilled in the non-draining sinks. This provided fairly satisfactory drainage, although a serious maintenance problem was created.

Since World War II, Fort Campbell has been converted into a permanent base that now houses the 101st Airborne Division. The old wooden buildings are gradually being replaced by more permanent structures.

PREVIOUS FOUNDATION FAILURES

The first building foundation failure occurred during construction of a concrete-frame masonry-wall barracks building. A small sink suddenly appeared inside the structure partially undermining a bearing wall. The problem this situation created was solved by underpinning the wall with seventeen cast-in-place piles. The Nashville District of the Army Crops of Engineers was not then concerned, with the result that its records are meager.

In 1956, the Nashville District of the Crops awarded a contract for construction of a fire and crash station to be built on a site on the adjacent air base. On the day that the notice to proceed with construction was to be issued, a hole 15 ft in diameter and 5 ft deep appeared in the middle of the site. The contract was cancelled, and the area of the new sink drilled. Fig. 1 represents the information the borings disclosed. The height of the open cavity penetrated by the drill exceeded 20 ft and the overlying soil was badly disturbed. As construction over such a foundation condition appeared unwise, a new site was located and drilled. When a site free of caves was found, the job was readvertised and built without trouble.

As a result of the experience at the fire and crash station, the subsequent foundation explorations for major structures at Fort Campbell have included

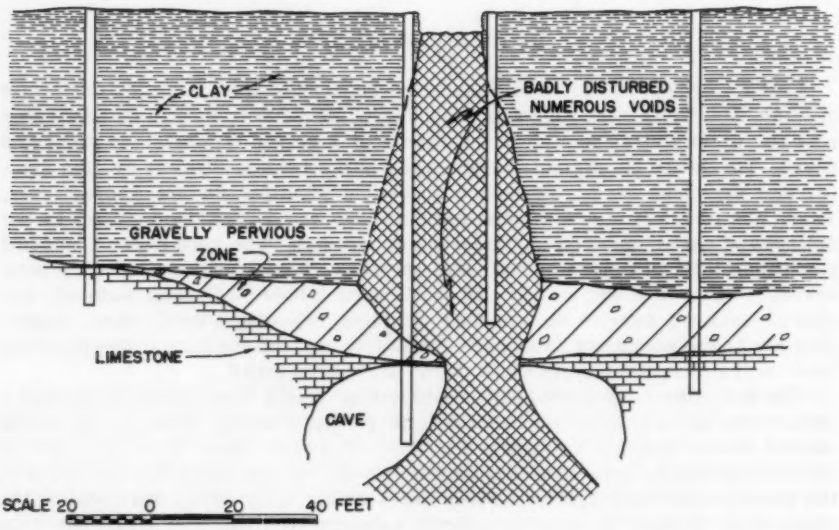

FIG. 1.—CROSS SECTION AT FIRE AND CRASH STATION SITE, SHOWING INTERPRETATION OF FOUNDATION CONDITIONS

some core boring in rock. It is impossible to avoid putting some structures over small caves, but it should be possible to avoid areas in which caves are large or very numerous.

For preliminary planning, the Corps has prepared an aerial photo-mosaic of the base from Department of Agriculture aerial surveys made in 1937, on which the present base layout is superimposed. Aerial photos made because the base was built would not serve the same purpose, as many sinks have been obscured by grading.

More recently, the Corps has also developed a master plan on which both the old and new sinks are shown. Boring locations are shown on this, and all areas that for any reason appear to have poor foundation conditions are out-

lined. This master foundation-information plan is a great help in searching for building sites with good foundation conditions.

SELECTION OF THEATRE SITE

The theatre building site with which this paper is concerned had been well explored (Fig. 2). Fourteen holes had been drilled and the location of the building shifted twice. The location chosen finally appeared to be the best in the immediate area. Near one end of the site, two holes penetrated cavities in bedrock: Hole 105, 8 ft of open cavity, Hole 126, 21 ft of clay-filled cavity. At the other end of the site, overburden thickness increased rapidly and there was an old sink not far away. Only two holes of the nine drilled under the final location, Holes 127 and 128, found cavities in rock. Both cavities were $3\frac{1}{2}$ ft high,

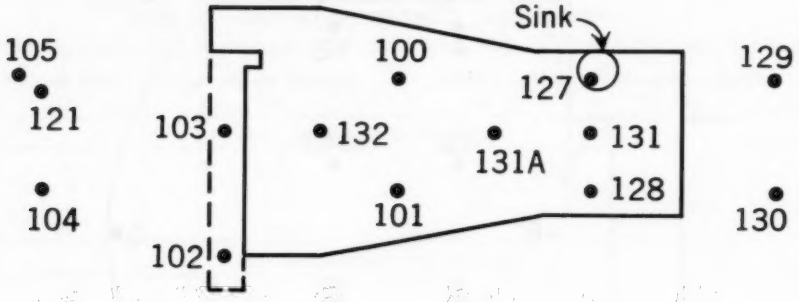

FIG. 2.—PLAN SHOWING BORINGS FOR THEATRE SITE

were filled with clay, and had roof covers of 5 ft and $7\frac{1}{2}$ ft of good rock, respectively.

APPEARANCE OF SINK AT THEATRE

Construction of the theatre was begun in March, 1957. On Friday, November 18th, the concrete frame had been finished and the roof trusses installed. The concrete stage was formed and ready to place when a heavy rain shut down work temporarily. By closing time, when the rain had stopped, it was found that surface run-off had flooded the excavation. On Monday morning the work crew, expecting to pump out the excavation, found it to be empty. A sink 12 ft in diameter and 3 ft deep had appeared under the front corner of the stage, and the water had drained into deep cracks in the side of the sink.

At this time, approximately $150,000 was invested in the theatre, but a sink in the middle endangered the structure. There appeared to be three courses of action possible:

1. underpinning the foundation for at least 40 ft or 50 ft around the sink;
2. grouting the sink; or
3. abandoning the site.

The choice between underpinning and grouting was a difficult one. Underpinning had been used to remedy the only previous failure of this type, and with success. Underpinning appeared, at first thought, to be a sure remedy, but it would have been very difficult to determine how much of the building required such support. Grouting also appeared a little uncertain. Just what kind of openings occurred was not certain, but grouting should help to determine this. Grouting was estimated to be cheaper, if it would work, but care would be necessary to obtain an adequate job without too much waste. It could be possible to pump fantastic quantities of grout into a foundation such as this. However, the core borings made at the site had not indicated extensive caves in the rock under the building, and it appeared that the trouble must be related to openings

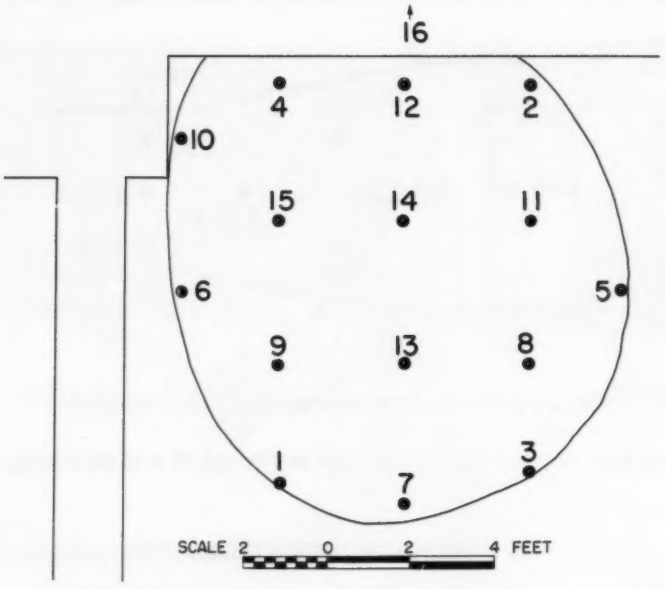

FIG. 3.—PLAN OF SINK SHOWING GROUT HOLES

in the overburden. If this were so, it was decided that it should be possible to grout the openings with cement grout, without wasting too much grout outside the area of the theatre's foundation.

The decision, therefore, was to attempt grouting, reserving underpinning for consideration only if the grouting seemed unsatisfactory.

GROUTING THE THEATRE FOUNDATION

Fig. 3 shows the pattern of grout holes in the sink. The holes were drilled and grouted successively in numerical sequence, 1, 2, 3, 4, and so on. The se-

quence used was designed to avoid drilling near a freshly grouted hole before the grout had set.

All grout holes were cased. Each hole was drilled to rock and cased to the bottom. If water would not run out of the casing set at the bottom it was pulled up until it did drain when grouting was started. If the hole stopped taking grout, the casing was pulled up until either the hole started taking grout again or the casing was completely removed.

A high speed grout mixer was used. This machine mixes grout on the centrifugal pump principle and will discharge it through several hundred feet of level line. The mixer pressure of approximately 5 psi, plus gravity, provided all the pressure necessary, and no grout pump was needed. Grout was conveyed to the hole through a fire hose. The relative pressure and rate of flow could be judged by feeling the hose near its connection to the casing. Thus, no pressure gage was necessary.

TABLE 1.—GRADATION OF SANDS USED IN GROUT

Sieve No. (1)	Percentage Passing (2)	Sieve No. (3)	Percentage Passing (4)
(a) Agricultural limestone			
4	100	50	24.0
8	71.1	100	16.6
16	45.7	200	9.2
30	32.3		
(b) Commercial mortar sand			
4	100	50	16.6
8	98.1	100	5.5
16	78.6	200	1.9
30	41.6		

Grouting was done with both sand-cement mortar grout and neat cement grout. There were two mortar grout mixes, each designed to use a particular sand that was available locally, because the job was too small to justify getting a special sand. Both sands were crushed limestone; the first was an agricultural limestone whose gradation, having an excess of rock flour or minus 200 mesh sizes, is shown in Table 1. This material gave a workable mix of 1 bag cement: 1.56 cu ft sand: 1.6 cu ft water. The other sand was a commercial mortar sand produced by scalping a well graded concrete sand on the No. 4 sieve. The gradation is indicated in Table 1.

The mix design for this sand was 1 bag cement: 2.22 cu ft sand: 0.8 cu ft water. The fact that more sand and less water was used to get a mix of equivalent workability (as tested with a funnel viscometer[2]) dramatically shows the advantages of using a well graded sand without an excess of rock flour. The better graded sand is obviously more economical, and not surprisingly, it produced a grout that was stronger when cured. The 28-day compressive strengths were 2829 psi and 4368 psi, respectively, for the two mixes.

[2] "Grouting Subballast for Track Bases," Bulletin, Portland Cement Assoc., No. ST 49.

The neat cement grout used had a water-cement ratio of one cu ft per bag that was not varied.

The first holes grouted were extremely open, with drill water draining freely over the entire area. Hole No. 1 lost drill water 10 ft above rock; hole No. 2, 14 ft; hole No. 3, 2.3 ft; and hole No. 4 at 9-1/2 ft. Hole No. 6 was the only other boring to lose all the drill water. These holes were initially grouted with sand-cement mix. As the area tightened up and holes took water less freely, neat cement was used. Grout travel was controlled by grouting intermittently, arbitrarily limiting the amount of grout in each hole to about 200 cu ft at a time, at which point drilling and grouting began on the next hole in the sequence. As the grout pattern became "filled in" and the area showed signs of becoming consolidated, the holes that had not refused were regrouted. All these refused grout after taking moderate amounts. With this procedure, it appeared that some grout may have traveled a hundred feet or more from the area being grouted, a condition that was anticipated.

The first four grout holes took 79% of all the grout that went into the sixteen holes. In sixteen holes, a total of 34 cu yd of grout was injected under the 12-ft diameter sink. Of all the grout injected 72%, by volume, was sand-cement mortar grout, that was used altogether at first. The last holes took so little grout that it seemed obvious that the area was well grouted.

EXPLORATION AFTER GROUTING

Although a pattern of gradual "tightening" of an area is good evidence of effective grouting, the success of the operation was also checked by twenty-two exploratory holes, of several types, as shown in Fig. 4. Only two of these holes, the angle holes shown at the top left of the figure, lost drill water in the overburden, although thirteen of the original fourteen exploratory holes had done so. In these holes, the overburden was grouted before they were cored into rock. After drilling through 12-1/2 ft of rock, the core hole at the corner of the building that was angled alongside the sink encountered a foot-high partially open horizontal cavity in rock. This cavity was subsequently grouted. These three angle holes took 21 cu yd of grout.

The following additional features were noticed in connection with the exploratory operation:

1. The holes outside the sink area revealed no trace of grout in the overburden. This may have resulted from closure of smaller openings as larger openings were successfully grouted.
2. The undisturbed samples from the sink area contained a good deal of grout, that was confined to seams running through a gravelly clay matrix. No grout was intruded into gravel.
3. The soil layers between the grout seams were not as firm as undisturbed ground, but laboratory tests showed that it was adequate to support the load of the building. The soil compaction percentage tested in accordance with American Association of State Highway Officials (AASHO) T-99 modified ranged from 65% to 84%, as compared to 70% to 89% for samples outside the sink area.
4. The overburden found in all samples was clay, with some gravel.

COMPLETION OF BUILDING

On completion of the consolidation grouting, exploration and soils testing, construction of the building was resumed. The sink area was cleaned out and

backfilled, steam and water lines crossing it were repaired. In two years, there has been no measurable settlement of the sink area, and the condition of the theatre building is as good as that of any other on the base.

CONCLUSIONS

The most important disclosures resulting from this grouting were; (1) determination of the character of the overburden at Fort Campbell, and (2) that openings in the overburden are not necessarily directly over cavities in rock. Originally, it was supposed that the pervious zone in the overburden is open gravel and that the theatre sink was the result of clay washing out through the

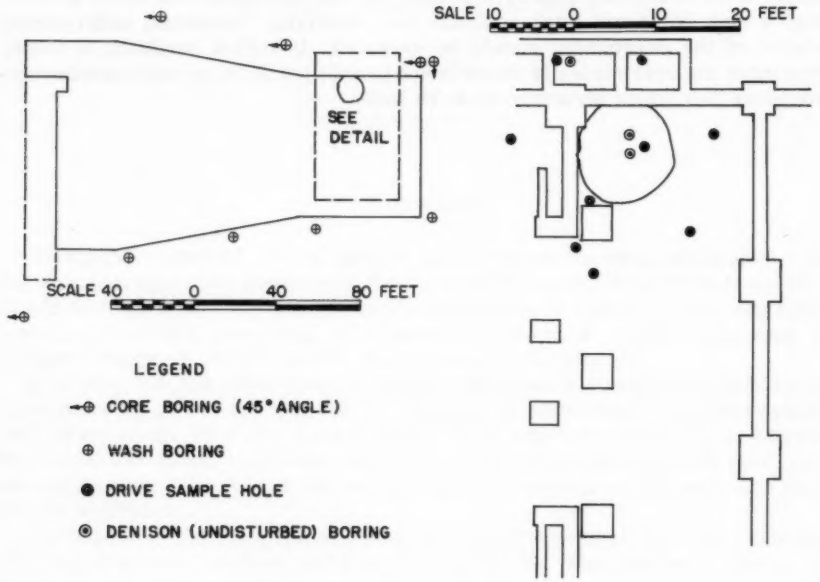

FIG. 4.—PLAN OF FOUNDATION EXPLORATION AFTER GROUTING

gravel. However, the exploration around the theatre sink showed no particular connection between the gravel and voids. Instead, there was apparently a zone of underground erosion with mud-filled openings and soft ground adjoining these. This is decidedly a worse condition than an open gravel stratum, because it greatly reduces the strength and stability of the foundation.

This weak zone, that was found in over half the holes drilled on the base, appears to be much more widespread than bedrock caves that are ready to collapse. So much of the base is underlain by this pervious zone that it is hardly possible to avoid building over it. However, buildings can be located away from the areas in which the pervious zone is thickest and shallowest. If a major structure must be located over a thick pervious zone, pregrouting of the over-

burden would give insurance against foundation failure during and after construction.

One measure that should reduce the amount of trouble from sink holes is to keep overburden excavation to a minimum, and this is now the policy at Fort Campbell. A second important ground rule for future construction calls for provision of adequate surface drainage away from the excavation.

The extreme difficulty of adequately exploring this type of foundation should be apparent. None of the various kinds of borings can show the true nature of the pervious zone. Undisturbed samples are generally disturbed by the presence of gravel, and small voids in soft materials collapse in the sampler. In this particular case, wash borings appear to give the most reliable information for the money available once the general foundation condition has been diagnosed.

This is not to suggest that the foundation conditions at Fort Campbell are typical of all limestone country, for they are not. Yet, wherever there are extensive sink holes and thick residual clay overlying limestone, underground erosion of the overburden should be expected. If such a condition is found, grouting of the overburden is feasible and probably will be the best stabilization procedure if a major structure must be built.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

SYMPOSIUM ON GROUTING: GROUTING TO PREVENT VIBRATION OF MACHINERY FOUNDATIONS

By John P. Gnaedinger,[1] M. ASCE

FOREWORD

This paper is one of several papers that constitute the Symposium on Grouting which occupied two sessions of the June 1960 meeting of ASCE in Reno, Nev. Credit for organizing this symposium program goes to a Special Grouting Symposium Committee consisting of Stanley J. Johnson, F. ASCE, Chariman, T. William Lambe, F. ASCE, and W. K. Seaman, F. ASCE.

It is believed that these several papers constitute a significant contribution to engineering practice in the art and science of stabilizing foundation materials by grouting. This is a field in which there appears to be increased interest among foundation engineers and one which is expected to become more important as time goes on and the need to build structures on difficult soil conditions increases.

The papers of the symposium are concerned with cement, clay, and chemical grouting of seams, cavities, and granular materials. The papers illustrate the increased use of chemicals for grouting purposes and also their use with more conventional grouting materials such as clay and cement; grouting of overburdened materials which is rapidly becoming a more definite art and science and which, to some degree at least, is susceptible to laboratory studies; the grouting of a cut-off in granular materials to prevent underseepage beneath a major hydroelectric project (perhaps the first application of this type of grouting in the United States); and the grouting of subsurface voids and solution channels in limestone areas subject to large subsidence.

Note.—Discussion open until September, 1961. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

[1] Pres., Soil Testing Services, Inc., Chicago, Ill.

Organized in 1952, the Grouting Committee with Task Committees on Cement, Clay, and Chemical Grouting has sponsored groups of papers or symposia occupying the sessions of meetings of the Society in 1954 and 1956, as well as 1960. Papers on grouting which have been published in the Journal of the Soil Mechanics and Foundations Division are as follows: Nos. 1145, 1204, and 1544 to 1552 inclusive. The report of the Task Committee on Chemical Grouting appears as Paper 1426 in the November 1957 Journal. A draft of a report on clay grouting has been prepared by the Task Committee on Clay Grouting, and the report on cement grouting prepared by the Task Committee on Cement Grouting is substantially in its final form.

The Administrative Committee on Grouting takes this opportunity of expressing its appreciation to the task committees for the excellent manner in which they have carried out their assignments and to the individuals who, as writers of papers, have made significant contributions covering the practices of grouting in various fields.

Raymond E. Davis, F. ASCE, Chmn., Admin. Committee on Grouting, Prof. of Civ. Engrg., Emeritus, Univ. of California, Berkeley, Calif.

SYNOPSIS

This paper describes a technique which has been successfully used on two projects to reduce the amplitude of vibrations of an expander and two compressor foundations to tolerable magnitudes. This is accomplished by injection of chemical grout into the soil supporting the foundations, thus effecting modification of the elastic properties of the soil.

Information on soil conditions at each site and data on vibration measurements before and after grouting are provided. It is felt that this solution to several vibration problems represents an additional tool in the hands of the foundation engineer in eliminating resonance in existing machine foundations. It also provides data which should be of value in enabling engineers to predict the natural frequencies of proposed soil-foundation machine systems.

INTRODUCTION

The science of soil mechanics has not found a sound solution to the problem of predicting the natural frequency of a vibrating machinery foundation. The natural frequency is that frequency at which the soil-foundation-machine system, when subjected to forced vibrations in any of the six modes of vibration, will vibrate with a maximum amplitude. This phenomenon of vibration is generally termed as resonance. The amplitude at resonance is usually much higher than the amplitude at higher or lower frequencies.

Soil and foundation engineers are actively predicting settlements of embankments and buildings with quite reasonable accuracy, and are improving the reliability of methods of determining the ultimate bearing capacity of various foundation and soil conditions. However, it is unfortunate that little information or technique is available that will permit an engineer to design properly and efficiently a machine foundation that will not vibrate excessively.

Perhaps the principle reason for the lag in this phase of the science of foundation engineering is that most machinery foundations do not vibrate excessively. However, the few that do vibrate are a constant nuisance to the owner of the machine and a constant thorn in the side of the machine manufacturer. The consulting structural or foundation engineer, strangely enough, is not usually blamed for the failure to anticipate the vibration problem, as long as he conforms to the manufacturer's empirical requirements, such as that the foundation for the machine shall be so proportioned that the soil pressure shall not exceed, for example, one-half or one-quarter of the allowable pressure for the particular soil. Other empirical requirements involve a ratio of foundation weight to either the horsepower or the speed of the machine.

It would seem apparent that the responsibility of the manufacturer should end when he has built a machine with a minimum possible vibration for the function performed. The foundation engineer should carry the foundation design to completion, considering the nature of the machinery vibration, the soil conditions, and the amplitude of vibration that can be tolerated by the owner or his neighbors.

The theoretical analysis of a vibrating machine-foundation-soil system is highly indeterminate, and most solutions that have been presented to date (1961) are based on assumptions which over-simplify the problem by neglecting certain important factors in the vibrating system, or otherwise give results that are not compatible with actual observations.

It appears probable that the greatest hope for a solution to vibrating machinery design problems lies in the analysis of actual cases of vibrating machinery foundations, including pertinent information on soil conditions and vibration characteristics.

EXISTING KNOWLEDGE ON VIBRATING MACHINE FOUNDATIONS

A rather thorough examination of the problem of vibrating machine foundations has been presented[2] by G. B. Tschebotarioff, F. ASCE. Additional study of the problem involved has been presented[3,4] by the American Society for Testing Materials (ASTM).

Historically, initial studies of the natural frequency of machine foundations were based on the assumption that a mass was supported on a spring, in which case the natural frequency is proportional to the square root of the spring constant and inversely proportional to the square root of the mass.

This relationship was later modified to take into consideration a damping constant, which has the effect of reducing the natural frequency from that computed for the freely vibrating linear motion of the mass on the spring. It is

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[2] "Soil Mechanics, Foundations, and Earth Structures," by G. P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, 1951.

[3] "ASTM Symposium on Dynamic Testing of Soils," Special Tech. Publication No. 156, 1953.

[4] "Vibrations in Semi-Infinite Solide Due to Periodic Surface Loading," by T. Y. Sung, ASTM Symposium, 1953.

recognized, however, that the damping constants for various soils are not known, and it is doubtful that the damping factor is actually a constant for a particular soil.

Recognizing that the soil beneath the foundation must also move with the foundation during vibration, it has been assumed that the natural frequency was inversely proportional to the square root of the weight of the vibrating machine and its foundation, plus a weight of soil. However, the weight of soil which should be considered is not clearly defined, and probably varies with the amplutude of the vibration.

Frequently machinery manufacturers recommend that the pressure beneath the machine foundation should be a certain fraction of the pressure which would be permissible for static loads on the soil present at the site. Such a procedure is not logical, since a particular soil type does not have a natural frequency as a basic characteristic property, nor is the natural frequency of the machine-foundation-soil system related necessarily to the strength of the soil beneath the foundation.

Various tables have been prepared giving the natural frequencies for various soil types. Generally the range, in cycles per minute, is from approximately 500 for soft organic soils, 1,100 to 1,700 for clays and sands, and 2,000 cpm or more for rock.[2] It has been pointed out that while the bearing capacities may vary from several hundred pounds per square foot to several hundred thousand pounds per square foot (for rock), the natural frequencies only vary a relatively small amount (500 cpm to 2,000 cpm) for the natural frequency of organic silt compared to that for bedrock. Measured natural frequencies have been noted outside this range but not appreciably so. Thus, it is all the more apparent that the natural frequency cannot be directly related to the bearing capacity of the soil.

It is believed that differences in the unbalanced force for a given machine will change the natural frequency, since a greater unbalanced force will presumably create movement in a larger mass of soil and thus reduce the natural frequency. This opinion has been substantiated by a limited number of tests.

Furthermore, for determining the equivalent spring constant for a foundation, it is probable that the spring constant will increase with increased amplitude of vibration, and furthermore, that the amplitude for a given soil type will vary with the loaded area. Studies (in 1953) have indicated that the pressure distribution beneath the vibrating foundation will also vary considerably with the frequency and amplitude of vibrations.[4]

Thus it appears that the natural frequency of a soil-foundation-system must vary with the following:

1. The soil or rock characteristics and stratification;
2. the weight of the vibrating equipment and its foundation;
3. the area;
4. the nature of the unbalanced forces and resulting vibrations; and
5. the effects of nearby surcharges or excavations.

As a further point, aside from resonance and its accompanying problems, the foundation engineer must also be concerned with the possible densification of soils beneath the foundation resulting from the vibration. It has been found that densification of granular soils can result from vibrations, but densification is not to be expected in cohesive soils. The amount of densification in sand obviously will vary depending on the original density of the sand, as well as the nature of the vibration.

Perhaps the best reference in the design of machinery foundations to avoid resonance is the chart showing the relationship between reduced natural frequencies plotted against the area of the foundation, with a band of values for various soil types, given by Tschebotarioff.[3] However, even after consulting this chart and designing accordingly, the designer should recognize that the solution is not a precise one, and should advise the owner accordingly. Tschebotarioff has suggested leaving cavities within the foundations which can subsequently be filled if necessary to change the weight and consequently the natural frequency of the foundation.

Since pile or caisson foundations probably provide elastic support comparable to that of a dense sand foundation, and since resonance has been observed on foundations supported on piles as well as on sand foundations, it should not be expected that the use of pile or caissons will automatically eliminate the possibility of resonance in the foundations.

CORRECTING RESONANT CONDITIONS

Statistically, most vibrating machinery foundations do not exhibit resonance at operating frequencies and are not a source of concern to the owner or nearby parties. This is certainly a fortunate coincidence, and probably occurs because of the great effort on the part of equipment manufacturers to minimize vibrations, and also because of the general tendency of foundation designers to provide a relatively massive foundation which will generally result in a relatively low natural frequency. However, many problems do arise where the operating frequency approaches the natural frequency of the foundation-soil mass system and resonance occurs resulting in considerable damage and nuisance, and in some cases actual danger.

When such problems of resonance have occurred, they have sometimes been solved by such techniques as separating the foundation from the adjacent floors, conversely tying the foundation to the adjacent floors, lowering the water table, filling cavities provided in the foundations, and adding mass by extending or underpinning the foundation.

The following describes a technique which involved injecting chemical grout into the soil beneath the vibrating foundation, thus changing the elastic properties of the soil and consequently changing the natural frequency of the foundation. In both cases described, the amplitude of the vibrations were reduced to tolerable amounts by the grouting program.

Vibrating Expander Foundation.—An expander foundation was operated for the first time in January 1958, before necessary piping connections had been made to the expander. Considerable vibration was noted in the machine. Soil borings immediately outside the expander room indicated a dense sand stratum extending to a depth of approximately 12 ft below the foundation. In addition, field density determinations made in the excavation for the foundation prior to construction of the expander indicated densities of approximately 100% modified American Association of State Highway Officials (AASHO) density in four separate tests. The water table was known to exist approximately 3 ft below the floor slab, which is approximately 2 ft above the bottom of the expander foundation.

Vibration measurements on the expander foundation, determined from acceleration and run-down tests, indicated that the natural frequency for a rotational (rocking) motion in the foundation was approximately 300 vpm, while the

operating frequency was approximately 280 vpm. With the machine thus operating at near resonance in one of the six possible modes of vibration, a solution to the problem appeared to lie in changing the natural frequency of the system, since the operating frequency could not be changed (Fig. 1).

Initial consideration was given to the possibility of adding concrete to the foundation. However, space limitations and existing trenches along the foundation presented obstacles that precluded this as a practical solution. Also, since a slight increase in weight of the foundation would further reduce the natural frequency, it was deemed possible that the addition of concrete could aggravate rather than correct the existing conditions because it would perhaps lower the natural frequency to coincide with the operating frequency.

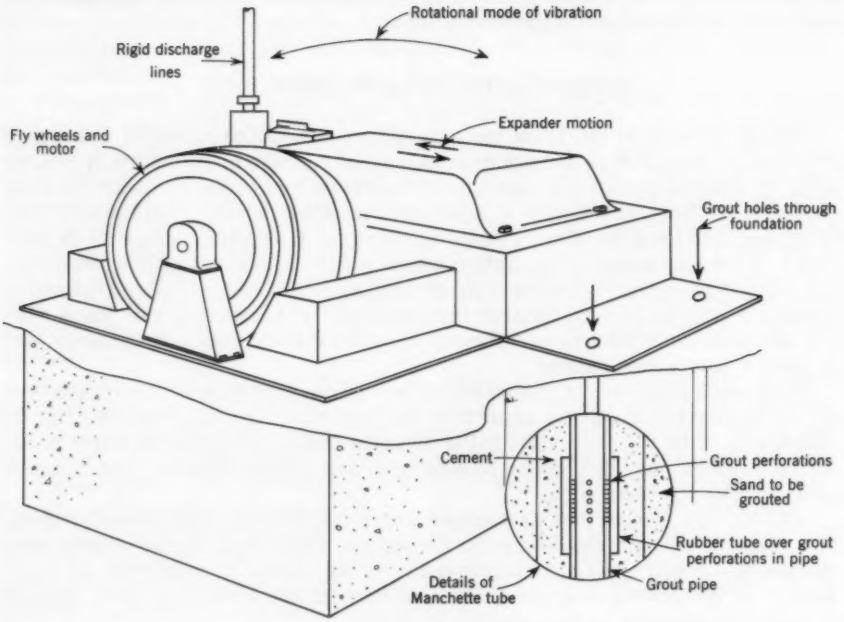

FIG. 1.—GAS EXPANDER AND FOUNDATION

Considerable study was given to the possibility of changing the nature of the unbalanced forces in the machine, though it was felt that the change in the direction of the unbalanced forces, which was the only change that could be made in the equipment, would not necessarily eliminate the objectionable vibration. Subsequently, however, some changes were made in the equipment simultaneously with the grouting program, thus complicating to a certain degree the evaluation of the grouting performance on this particular project.

A further consideration on this project was that grouting the sand beneath the foundation would effectively prevent densification of the sand due to the vibrations of the machine. Since settlement in the expander would be rather critical because of the rigid pipe connections and high pressure used in the system,

it was felt that the effects of grouting would be beneficial in preventing subsequent settlement, aside from reducing the displacements due to vibrations.

Support of the foundation on piles or caissons carried to rock was also considered, though major construction complications were involved in such procedures.

Prior to proceeding with field grouting operations, a testing program was initiated to determine by laboratory tests the effect of several chemical grouts on the elastic properties of the sand. Although it was realized that dynamic tests would be perhaps preferable, it was felt that the cost of such testing would be prohibitive.

Tests were performed using both acrylamide methylene bisacrylamide (AM -9) and sodium silicate as grouts.

Triaxial test results indicated an appreciable increase in cohesion due to each of the chemical grouts, and the stress-strain curves for a lateral pressure of 5 psi indicated that each of the chemically solidified samples displayed a distinct increase in the slope of the stress-strain curve to a value approximately 3.8 times the value for the natural sand.

On the assumption that the dynamic modulus would increase in the same proportion as the static modulus (as determined by the triaxial tests), it was expected that the natural frequency of the foundation might be increased by a factor of 1.86, or from a frequency of 300 cycles per minute to 558 cycles per minute, which might then greatly reduce the amplitude of the vibrations.

In order to perform the grouting operation, the holes were drilled at each end of that portion of the foundation where the horizontal motion of the expander was causing the rocking motion in the foundation.

After drilling these four holes through the concrete foundation, casing was driven to a depth of approximately 8 ft below the bottom of the foundation. The sand within the casing was washed out, and a 2 in. diameter pipe with perforations which were covered with rubber sleeves (a Manchette tube) were lowered into the casing and then surrounded by a weak cement-bentonite grout. Before the grout set, the casings were removed with the assistance of a crane located within the structure. After the cement grout "set-up," a packer and grout pipe arrangement was then inserted inside the 2 in. pipes. After the packer was set at each predetermined depth, grout pressure through the grouting pipe expanded the rubber sleeve and fractured the grout seal around the 2 in. pipe. The AM-9 grout was then forced into the sand surrounding the pipe. This procedure was repeated at each of the four grouting holes and at 1 ft intervals in each pipe.

Approximately 550 lb of AM-9 grout at 10% concentration were used in each of the injection pipes. Although the exact distribution of the grout within the soil mass was not determined, the amount of grout used was such as to create a solidified mass of soil contained in a right circular cylinder approximately 4 ft in diameter and 8 ft deep surrounding each of the injection points.

Subsequent measurements of the vibrations in the machine indicated that the amplitude of the vibrations had been reduced to approximately one-third original values, though the nature of the vibrations in each of the six modes changed somewhat during the operation, partially due to the effects of the grouting and partially due to modifications in the equipment made by the manufacturer. After the grouting, the natural frequency was determined to be 402 vpm, in contrast to the original natural frequency of 300 vpm.

Y-Type Compressor Foundations.—Several months after the completion of the expander foundation project, a similar problem arose in two compressor

foundations in Lynchburg, Va. At this installation, Y-type air compressors, rated at 225 bhp each, were installed on a 12 ft by 7 ft by 2 ft-6 in. thick reinforced concrete slab. After installation, the compressors were found to vibrate with objectional amplitude. The amplitude of these vibrations was considered to be sufficient to cause possible crystallization of piping joints and reduction in the life of bearings in the compressors. In addition, the vibrations

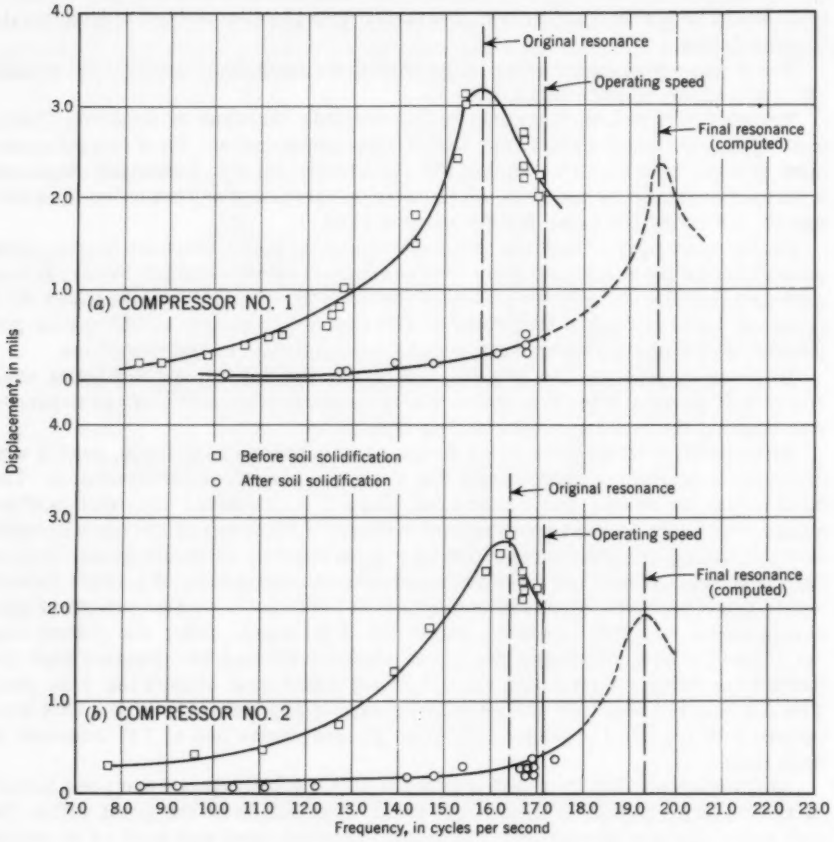

FIG. 2.—LONGITUDINAL VIBRATORY DISPLACEMENT VERSUS FREQUENCY

were being transmitted through the soil to adjacent portions of the boiler room and causing considerable annoyance and minor damage.

A seismograph analysis (Figs. 2 and 3) indicated that the natural frequency of the foundation initially was 948 cycles per minute, while the frequency resulting from the operating speed was approximately 1,028 cycles per minute. It should be noted that the vibration frequency at resonance was double the operating speed of the compressors, indicating that resonance occurred at the

second harmonic of the operating speed. The analysis further indicated that the vibration was in a rotational mode about an axis parallel to and 6 ft-3 in. below the compressor crankshaft. The translational movements and the movement in the other rotational modes was found to be insignificant.

As a result of the vibration analysis and the conclusion that the vibration was primarily in a rotational mode, the stabilization was proposed beneath the

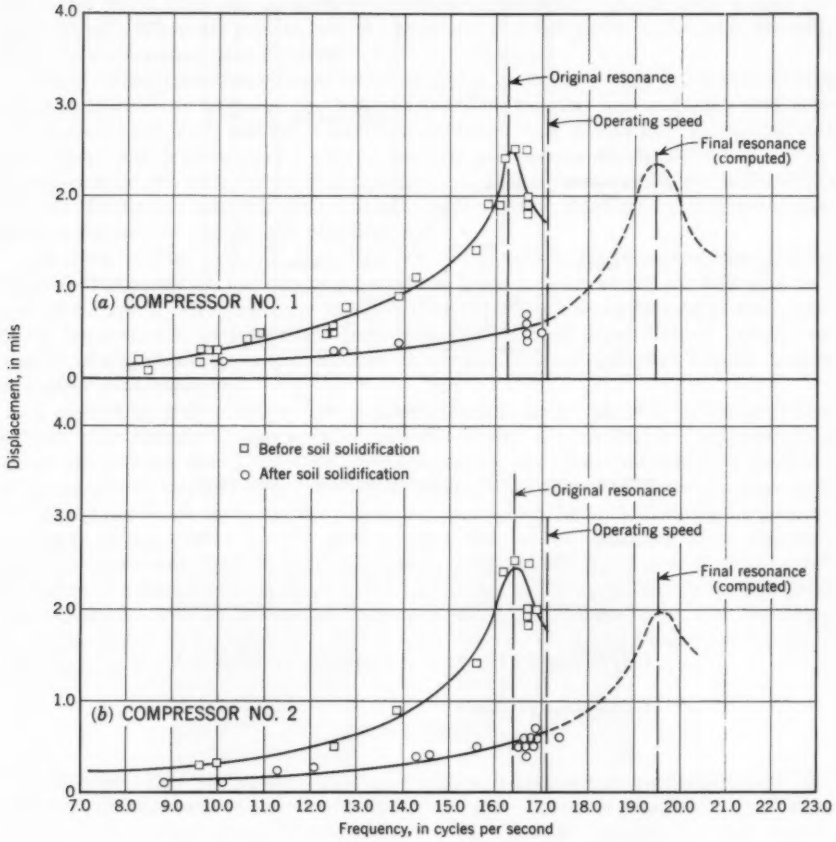

FIG. 3.—VERTICAL VIBRATORY DISPLACEMENT VERSUS FREQUENCY

ends of the foundation slab to a maximum depth of 7 ft-6 in. below the bottom of the slab. Termination of the stabilization at this elevation was based on the assumption that the effect of the vibratory loading would be negligible at this depth and any attempt to change the natural frequency of the vibrating system by stabilization beyond this point would be uneconomical. Fig. 4 is a plan showing theoretical limits of the designed stabilization.

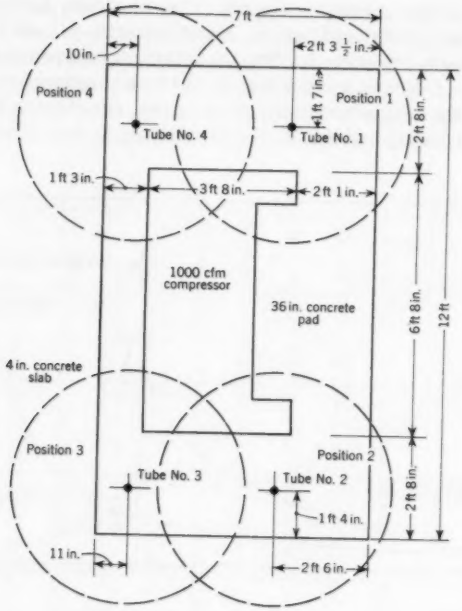

FIG. 4.—PLAN OF COMPRESSOR FOUNDATION

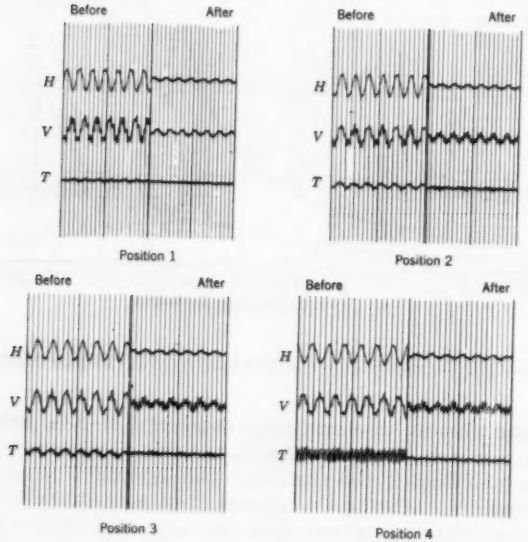

FIG. 5.—COMPARISONS BETWEEN VIBRATION MEASUREMENTS MADE BEFORE AND AFTER GROUTING

The field operation involved the removal of 4 in. cores from the slab at the locations indicated in Fig. 4 and augering 4 in. holes to a depth of 7 ft-6 in. below the bottom of the slab. The soils at this location were found in a previous soil investigation to consist essentially of medium dense to dense sandy silts with a trace of clay, with no evidence of ground water in most of the borings.

After augering 4 in. holes to a depth of 7.5 ft below the bottom of the slab, Manchette tubes, 2 in. outside diameter, with rubber sleeves covering the perforations were then placed in these holes and grouted in place with a neat cement grout. When all four tubes had been set and the grout had cured, the stabilization operation was started.

Prior to the injection of any chemical grout, a cement-clay grout was forced into the inter-face between the base of the slab and the soil in order that contact between the slab and the stabilized soil would be complete. Following this operation, AM-9 grout was injected into the micaceous sandy silt, under pressures consistent with overburden conditions, at the various stages indicated in Fig. 6. Sufficient quantities of chemical grout were injected to produce a stabilized volume of the design dimensions.

As seen in Fig. 2 and 3, stabilization of the soil in this manner changed the natural frequency of the vibrating system from a near-resonant 948 vpm to a non-resonant 1,176 cycles per minute, thus reducing the amplitude at the operating frequency to values that were less than 40% of those before treatment. Fig. 5 is a photostatic reproduction of original seismographic traces before and after stabilization.

It should be noted that on the compressor foundation project the natural frequency was changed from a value slightly below the operating frequency to a value moderately above the operating frequency. Because the effect of increase in mass due to the grouting operation would be to reduce the natural frequency, and since previous tests on the expander foundation project indicated a definite increase in the elastic modulus of the soil due to the grouting, it is apparent that the increase in the natural frequency was essentially a result of a higher modulus of elastic deformation in the sandy silt soils resulting from the grout stabilization. The coincidental increase in the unit weight of soil resulting from the grouting apparently was a minor factor, probably having the effect of diminishing the effect of the increase in the modulus.

CONCLUSIONS

It is hoped that soil and foundation engineers will develop a technique which will permit them to predict natural frequencies in vibrating foundation-soil systems and therefore to modify the foundation design to insure that the natural frequency is significantly higher or lower than the operating frequency of the machine.

However, where existing machines vibrate with excessive amplitudes, and when the soils on which the foundations are supported are sufficiently pervious to permit the introduction of chemical or cement grouts, then the natural frequency of the soil-foundation-machine system can be changed by the grouting and displacements due to the vibration can be reduced to tolerable amounts. It is further believed that the principal effect of the grouting is to change the

elastic properties of the soil, increasing the modulus of deformation, with a consequent increase in the natural frequency of the foundation system.

ACKNOWLEDGMENTS

The project for National Cylinder Gas Company (expander foundation) was performed under the supervision of Kenneth Greene. The project for General Electric Company (compressor foundation) was conducted under the direction of the Real Estate and Construction Operation of the General Electric Company. Vibration measurements on both projects and vibration analyses were made by Jack F. Wiss, M. ASCE, of Wiss and Associates, in Chicago, Ill. Grouting operations were performed by Chemject Corporation, Niles, Ill. under the supervision of James A. Zurbrigen.

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SYMPOSIUM ON GROUTING: RESEARCH IN FOUNDATION GROUTING WITH CEMENT

By Thomas B. Kennedy[1]

FOREWORD

This paper is one of several papers that constitute the Symposium on Grouting that occupied two sessions of the June 1960 meeting of ASCE in Reno, Nevada. Credit for organizing this symposium program goes to a Special Grouting Symposium Committee consisting of Stanley J. Johnson, F. ASCE, Chairman, T. William Lambe, F. ASCE, and W. K. Seaman, F. ASCE.

It is believed that these several papers constitute a significant contribution to engineering practice in the art and science of stabilizing foundation materials by grouting. This is a field in which there appears to be increased interest among foundation engineers and one which is expected to become more important as time goes on and the need to build structures on difficult soil conditions increases.

The papers of the symposium are concerned with cement, clay, and chemical grouting of seams, cavities, and granular materials. The papers illustrate the increased use of chemicals for grouting purposes and also their use with more conventional grouting materials such as clay and cement; grouting of overburdened materials which is rapidly becoming a more definite art and science and which, to some degree at least, is susceptible to laboratory studies; the grouting of a cut-off in granular materials to prevent underseepage beneath a major hydroelectric project (perhaps the first application of this type of grouting in the United States); and the grouting of subsurface voids and solution channels in limestone areas subject to large subsidence.

Note.—Discussion open until September, 1961. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

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Organized in 1952 the Grouting Committee with Task Committees on Cement, Clay, and Chemical Grouting has sponsored groups of papers or symposia occupying the sessions of meetings of ASCE in 1954 and 1956, as well as 1960. Papers on grouting which have been published in the Journal of the Soil Mechanics and Foundations Division are as follows: Nos. 1145, 1204, and 1544 to 1552 inclusive. The report of the Task Committee on Chemical Grouting appears as Paper 1426 in the November 1957 Journal. A draft of a report on clay grouting has been prepared by the Task Committee on Clay Grouting; and the report on cement grouting prepared by the Task Committee on Cement Grouting is substantially in its final form.

The Administrative Committee on Grouting takes this opportunity of expressing its appreciation to the task committees for the excellent manner in which they have carried out their assignments and to the individuals who, as authors of papers, have made significant contributions covering the practices of grouting in various fields.

Raymond E. Davis, F. ASCE, Chmn., Administrative Committee on Grouting, Prof. of Civ. Engrg., Emeritus, Univ. of Calif., Berkeley, Calif.

SYNOPSIS

Numerous investigations involving cement grouting of foundations have been conducted since the first large-scale grouting attempt in 1910. Modified cements, mixtures of cement and additives or replacement materials, and proper proportioning of grout ingredients have been developed for various projects as needed. Because the grouting requirements of each job differ, no all-purpose grout or grouting method has been established. However, results of the laboratory and field tests described in this paper show the advantages and disadvantages of cement grout incorporating admixtures of varying water-cement ratios and of varying grading and fineness of grout ingredients. In addition, grouting techniques, grouting pressures, and types of equipment utilized in these tests are described.

INTRODUCTION

Foundation grouting is used primarily in the treatment of rock formations and, to a lesser extent, for soil, sand, or sand and gravel formations. The over-all plan for a grout treatment is governed by the requirements of the particular structure. However, the primary requisite for any treatment is that the grout mortar readily and solidly fill the areas to be grouted and permanently retain its original volume. Standard grouting techniques often must be modified to meet existing conditions, and selection of proper materials, equipment, pressures, and techniques, as well as close control of proportioning of the ingredients of the grout, determine the success of a grouting job. Of course, a thorough knowledge of the foundation characteristics is a prerequisite to intelligent design of a grout treatment.

As a result of the diversity of problems that have been encountered in grouting work, numerous materials, mixes, techniques, and equipment have been developed. This paper presents a review of the literature relating to these developments.

RESEARCH IN CONJUNCTION WITH SPECIFIC CONSTRUCTION PROJECTS

Early Investigations at Estacada Dam.—The large-scale treatment of a foundation by grouting was first attempted in 1910 in the cutoff for the Estacada Dam (1).[2] The job required much experimenting, and although the grouting was considered unsuccessful, considerable knowledge was gained from the experience.

1. All drilling, testing, and grouting should be done through casings set in the concrete cutoff.
2. All testing should be performed from elevated tanks and not by pump.
3. Each hole should be tested and grouted as soon as drilled, then the drills kept away from the probable zone of diffusion for a few days.
4. In grouting, especially at high pressures, it is best to close the valve before the tank is entirely empty to prevent the air from following the grout into the hole.
5. A comparatively thin mixture should be used to begin grouting and, if taken freely, thickened until each succeeding batch requires either an increased discharge time or increased pressure. The forcing of charge after charge of thin grout into a hole probably is a waste of cement.

The experience at Estacada indicated that grouting should not be relied on in lieu of the usual concrete cutoff, for two reasons:

1. The efficiency of grout as a curtain wall cannot be predicted.
2. The proper diffusion of the grout can be secured only when the concrete of the cutoff closes the surface seams and confines the pressure to a depth at which it may be effective in tightening the underlying material.

Investigations at Hoover Dam.

Initial Work.—In 1933 Byram W. Steele, M. ASCE (2) outlined a program of investigations to obtain information for use in contraction-joint grouting at Hoover Dam. A review of contraction-joint grouting experiments was also included. The investigations were conducted in the Denver, Colorado laboratories of the Bureau of Reclamation, United States Department of Interior (USBR) and involved experiments with fineness of the cement, water-cement ratio of the grout, width of contraction-joint opening, grout pressure within the joint, and shearing strength of the grout film. For the investigations, a split cylinder of mass concrete, 5 ft in diameter and 7 ft high, cast to simulate a contraction joint in a dam, was so constructed that the thickness of grout film could be easily and accurately controlled. Although the laboratory tests were concerned with contraction-joint grouting, the following results are also pertinent to foundation grouting:

1. To obtain satisfactory flow in grouting a contraction joint, the cement should be passed through a 200-mesh sieve immediately before the grout is mixed.

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[2] Numerals in parenthesis, thus (1), refer to corresponding items in Appendix Bibliography.

2. For cracks of the order of 0.05 in., a grout in which the ratio of water to cement is 0.75 by volume gives the most desirable film. A water-cement ratio of 1.00 contains too much free water that cannot be forced into the concrete or otherwise dissipated.

3. Joint pressure of 90% efficiency and joint coverage of 99% are obtained easily with a grout having a water-cement ratio of 0.75, and with a film thickness of the order of 0.015 in.

4. Shear tests on grout films fog-cured at 70 F for 28 days demonstrated that increasing the pressure within the joint immediately after it is filled does not appreciably increase the shearing value of grout films of equal initial water-cement ratio.

5. With an apparatus used for determining punching shear, an average shearing value of 28 days age of films fog-cured at 70 F was 900 psi.

Hoover Dam (formerly Boulder Dam) was completed in 1936 and, on the basis of a great deal of experience obtained at this project, V. L. Minear, F. ASCE, and P. A. Jones (3),(4) reported certain conclusions as to the best materials, equipment, and procedure for grouting projects of a similar nature.

1. Grout should be injected by pumping and not by compressed air. Mixers should be provided with meters for controlling the amount of mixing water. (Grout equipment developed for this project consisted of a grout mixer, a mechanically agitated pump, and two high-pressure sludge pumps.)

2. The water-cement ratio, pressure, and rate of pumping should be coordinated by "feeling out" the hole to be grouted.

3. Pumping speed should be controlled by manipulation of the water-cement ratio. When the speed falls below normal, the water-cement ratio should be increased proportionately and vice versa.

4. Neat portland-cement grout is probably the best all-purpose grout and was used almost exclusively on the Hoover project for a sanded mixture proved undesirable because of its effect on the pumps as well as the difficulty of holding the sand in suspension.

5. As the use of extra finely ground cement is essential in contraction-joint grouting, it is of secondary importance in rock grouting.

Use of Special Cements and Additives.—Additional grouting at Hoover Dam during the period from 1938 to 1947 required further investigation of grout mixtures and techniques. A. W. Simonds, F. ASCE, and O. E. Boggess (22) described this program of construction which included the grouting of foundations and abutments, and extension of the drainage system. During the initial filling of the reservoir, particularly in 1937 and 1938, excessive seepage into the penstock tunnels developed and water from the foundation system entered the galleries of the dam in excessive quantities at certain localities. Large flows of cold water were discharged from several drains, and hot alkaline water dripped through cracks in the concrete lining of some of the penstock tunnels and damaged the paint and metal work. The uplift pressure on the base of the dam had increased by 1937 to undesirable magnitudes in certain areas. Consequently, the purposes of the additional grouting were to reduce this uplift pressure and seepage through the abutments. The grouting treatment to be described proved successful for these purposes.

The use of special cements for grouting the cracks leaking the hot alkaline waters, which occurred in warm spring areas, was investigated. Flash sets

occurred with portland cement, and the grout pumps would stall so that no grout was forced into the foundation as the pumping pressure built up. It was theorized that the flash sets were due to the high temperature of the alkaline water, the insufficient flushing of grout holes with cold water before the grouting began, or the contact of the cement with the alkaline waters.

The first special cement tried was an oil-well-type grouting cement which had been used successfully to grout areas in which hot salt water had been encountered. The oil-well cement was found to possess some desirable characteristics for grouting but was not completely satisfactory because it contained some unground clinker ("spitzers") which had a tendency to settle out when thin mixes were used.

Several cements were also tried during the program of additional grouting but did not give satisfactory results. Fly ash was tried with modified cement but produced no material advantage. At that time, the staff of the Concrete Laboratory in the Chief Engineer's office had been experimenting with modified cements and retarders in an effort to duplicate the characteristics of the oil-well cement. It was found that the addition of small amounts of a commercial retarder to modified (similar to present type II) cement resulted in a product that had characteristics similar to oil-well cement and was more finely ground. The retarder was composed of 87-1/2% calcium salt of lignin sulfonic acid and 12-1/2% triethanolamine. Experiments with the retarder at Hoover Dam produced beneficial results in the grouting. The retarder was mixed into a 12-1/2% solution and added to the mixing water of each batch in the proportion of one pound of retarder to six sacks of modified cement. Field tests indicated that it was possible to inject more cement into the foundation by using modified cement with retarder than by using either oil-well cement or modified cement without retarder. Retarder was used with modified cement in the grouting until May 1944 when the grouting had reached the areas of cold ground water and the retarder was not needed.

Effect of Water-Cement Ratio.—Experience had shown that grout having a water-cement ratio less than 3:1 by volume could not be injected at pressures greater than 450 psi without risking the plugging of the hole being grouted. Thinner grout mixes were more successful for squeezing off seepage through the andesitic foundation rock. At the beginning of the program of additional grouting, the water-cement ratio of the grout mix varied from 7:1 to 3:1 by volume. Thinner grout mixes were used more frequently as the work progressed; the most generally used ratios varied from 10:1 to 4:1. In fairly tight water-bearing rock, many holes were grouted with mixes having water-cement ratios of 20:1, 14:1, and 12:1, and unless surface leaks developed, grout thicker than 7:1 was rarely used. Thin mixes injected at pressures of 500 psi or slightly greater formed excellent grout films.

Grouting Procedure.—A systematic procedure of stage grouting was used at Hoover Dam. In areas in which open seams were crossed, as determined by the flow of water into the hole, it was customary to drill slightly beyond the seam and then grout it. After the grout had set for 16 hr, the hardened grout was drilled from the hole. The condition of the grout determined the type of bit to be used in cleaning out the hole. To save the diamond bits whenever possible, "stoodite" saw-toothed bits were used. In holes where flows of water of 200 gal to 400 gpm were encountered, a satisfactory cup, washer-type packer and a method for inserting it against large flows of water were developed. In areas where warm water was encountered, injection of fairly

large quantities of cold water into the grout holes for several hours before the start of grouting was found to be beneficial in retarding the time of set of the cement in the grout.

Studies of Grout Mixtures at Chief Joseph Dam.—In January 1950, J. M. Wells (23) described an experimental grouting investigation for Chief Joseph Dam, in which a method of grouting for the control of seepage into the excavation areas of the right abutment was studied. The program involved development of (1) a grouting mixture capable of penetrating the materials of the pervious formation; (2) an economical method for drilling a large number of holes in the gravel; and (3) a procedure for introducing the grout mixture into the pervious material. The efficacy of the grouting was also determined. Both laboratory and field studies were made.

Information on the formation of the right abutment materials prior to the grouting experiment raised a question as to whether cement grout would have the necessary penetrating qualities. Accordingly, several grout mixtures using bentonite, silt, plaster of paris, asphalt emulsion, and so forth, were developed in the laboratory.

Field Experience.—In the initial field trials, the portland-cement grout appeared to be penetrating the formation at Chief Joseph Dam successfully; consequently, it was used in the grouting program rather than the grout mixes with the other materials mentioned.

Several weeks were spent in attempts to jet pipes into the gravel formation but all attempts failed. A heavy churn drill rig was finally used successfully but progress was limited to about 10 ft per shift. The use of a well-graded sandy gravel (-3/4 in.) for backfilling drill holes around the grout pipes was very effective in preventing grout from flowing up the outside of the grout pipe casing.

A test pit excavated in the grouted area showed that the gravel was well sealed and effectively grouted from a depth of 23 ft to 52 ft at which depth bedrock was encountered. Areas of excessive leakage occurred in the upper zone where little attempt had been made to grout and at bedrock where cracks and fissures were not sealed. The following was considered an effective grouting procedure: (1) Grouting from bedrock up using 5-ft stages, (2) alternating between grout holes, and (3) using an initial mix of one part cement to five parts water with gradual thickening until a pressure of 150 psi was obtained. A grout-hole spacing of 7.5 ft produced an effective seal and results indicated that a greater spacing could be used. However, because of the excessive estimated cost for a grout curtain, the grouting scheme was abandoned in favor of an alternate plan providing for seepage control.

Foundation Grouting at the Savannah River Project.—In 1952 Stanley L. Johnson (26) reported on foundation grouting operations at the Savannah River Plant of the Atomic Energy Commission. The Atomic Energy Commission, the Du Pont Company, and the Corps of Engineers had collaborated in the geological and foundation exploration program for the Savannah River Plant and after careful study of the geological report it was concluded that foundation conditions were, in some respects, unusual and that grouting would be required beneath some of the structures.

The presence of sinks, due to solution of calcareous materials, throughout the plant area caused considerable concern for the foundation stability, and a foundation-grouting program was proposed. A limited number of laboratory tests were made at the Waterways Experiment Station, Corps of Engineers,

to determine the most suitable grout mixtures. Then a small-scale grouting test was made at the plant site to determine if the grout mixture proposed for use could be pumped satisfactorily or would segregate excessively. The test indicated that the foundation would freely take the grout pumped into it.

The foundation soils at the plant site consisted of sands and mixtures of clay and sand. Under normal circumstances, soils of these types cannot be grouted, even with a grout consisting of cement and water, and would never be considered capable of receiving a grout consisting of sand, cement, and water. But the highly unusual condition, that is the solution of the calcareous material in the zone beneath the plant, made it possible and necessary to grout. The grouting program accomplished the objectives desired and, in general, worked out as anticipated, although considerably more grouting was required than had been expected.

In the laboratory, various proposed grout mixtures were tested for tendency toward segregation and general workability of the mix. A few unconfined compression tests were performed to determine if the samples had adequate strength. It was considered unnecessary for the grout to have great strength, or that its rigidity should greatly exceed that of the natural ground.

Trial batches of grout were prepared using 8 cu ft of sand, 25 lb of bentonite, cement varying from 1 to 1-1/2 bags per batch, and water varying from 4 cu ft to 10 cu ft per batch. Other test batches were made up using the same proportions but substituting 1-1/2 cu ft of fly ash plus 2-1/2 lb of a water-reducing, expansive admixture for the bentonite. Tests indicated that the mix consisting of the 8 cu ft of sand, 1 cu ft of cement, 25 lb of bentonite, and 8 cu ft of water appeared satisfactory and it was used for the initial field tests. However, it was discovered that this mix set up too slowly, was weaker than considered desirable, and was too soft for testing the day after placing. A more suitable mix was obtained by increasing the portland cement to 1-1/2 cu ft and decreasing the water from 8 cu ft to 6 cu ft. The mixes consisting of sand, cement, admixture, and fly ash did not appear to possess any advantages over the latter sand-cement-bentonite grout. Therefore, after consideration of the availability of materials and cost, it was decided to use the sand-cement-bentonite grout in the field-grouting operations.

Other laboratory tests consisted of substituting slag cement for 50% of the portland cement in the standard mix and determining unconfined compressive strength on the resulting mixes. These mixes showed a tendency for a lower compressive strength at the early ages, but after 7 days, the strength was equal to or higher than that of the standard mix containing all portland cement.

Two series of tests were made with aluminum powder added to the grout mixture. In the first series, 4 g of aluminum powder was added to a batch of grout, and in the second series, 8 g was added. Compression tests revealed that the strength was not affected to any significant degree by the addition of aluminum powder. Tests were also made of the aluminum-powder mix to determine shrinkage and expansion characteristics. After 24 hr the plain grout had contracted 2.3% of its original volume (980 cc) and the aluminum-powder grout had expanded 4.3% and further expansion was indicated. The addition of the aluminum powder had no significant effect on segregation.

Unconfined compression tests were made with sawdust added to the normal grout mix in the proportions of 3%, 5%, 8%, and 10% by weight. The sawdust was coarse pine material, air-dried, and was mixed with the grout as it came from the field mixers. The compressive strengths of the sawdust mixes were

substantially lower than those of the standard grout mixes. However, a comparison of test data using the relation previously established between strength and water content indicated that the results of tests with sawdust mixes agreed with those of the plain grout for comparable water contents.

Use of Rock Flour at Norris Dam.—In describing the foundation treatment of Norris Dam during the construction period of 1933 to 1936, James S. Lewis (6) reported the results of some laboratory grout tests supplemented by additional investigations in the field. Preliminary field tests were made to determine the suitability of rock-flour mixtures for grouting. The results indicated that grout consisting of equal parts of cement and rock flour and a water-cement ratio of 1.0 was more desirable than plain cement grout because it penetrated the cracks better and did not set up in the pipeline during pumping. The rock flour was finer than the cement and definitely had a retarding effect upon time of set of the mixture. However, the use of a slow setting material such as rock flour increased the distance traveled by the grout fluid so that areas completely outside of the region that needed treatment were grouted and the quantity of material used was increased appreciably.

In order to reduce unnecessary consumption of the material, tests were made to determine the effect on setting time of adding calcium chloride ($CaCl_2$) to the grout. The acceleration of set caused by the addition of 3% $CaCl_2$ by weight of the cement to a marked extent counteracted the retardation caused by the rock flour. The resulting product possessed pumping and handling characteristics similar to those of regular portland-cement grout. Compressive tests on specimens removed from the mixer in the field showed that the product was sufficiently strong and would be able to resist erosion. Specimens cored from grout-filled seams after the rim grouting was under way exhibited a compressive strength of 2000 psi at approximately 45 days.

The rock flour used in these mixes contained a quantity of fine clay lumps that failed to disintegrate in the course of mixing. After some experimenting, it was found that a satisfactory method of breaking down these lumps was by use of a separate, mechanically agitated mixer from which the rock flour passed as a slurry through a screen to the grout mixer. The rock-flour mixer was similar to that used for the grout and was driven by the same type of air motor, but had one compartment instead of two.

During the preliminary grouting for sealing the first cofferdam, an attempt was made to economize by mixing sand with the grout. The experiment was unsuccessful as the pump and lines were plugged solid. Cores of the sand grout later removed from nearby holes showed a tendency toward segregation. The material in the cores was lean and crumbly and generally of poor quality. Lewis stated that he felt it would be unwise to attempt the use of sand for grouting seams in a foundation that would be subjected to more than a moderate head.

Cement Grout with Asphalt Emulsion.—As a result of a cooperative research and field testing program undertaken in 1941 by the Texas Company and the Atchison, Topeka, and Santa Fe Railway, and reported in 1946 by R. R. Thurston (14), it was discovered that in road-bed stabilization the addition of emulsified asphalt to the grouting mixture improved its penetrating and sealing properties and permitted the use of grout mixtures containing less portland cement. The standard emulsified asphalt originally used set too slowly, and a quicker-setting emulsion was developed by adding small amounts of chemicals to facilitate mixing with the cement and sand, to control the setting time, and

to minimize the amount of foaming and settlement. This emulsion was most successful in improving stabilization, lowering costs, and expediting grouting work.

Grout containing the quick-setting asphalt can be injected by either pneumatic or hydraulic pressure. Both methods have their advantages and both have been used successfully. Soil conditions determine the correct grouting mixture to be used. The recommended limits presented in Table 1 allow leeway for all types of conditions.

A very fine "blow" sand was found to be the most suitable for the grout mixture. Such a sand, of which about 90% will pass through a 48-mesh sieve and about 20% through a 100-mesh sieve, pumps most easily, causes the least wear on the pumping equipment, and flows most readily into the voids. When sand is not ideally graded in particle sizes, the addition of fly ash at the rate of 3 pcf to 12 pcf of other dry materials improves pumpability.

Grouting Lignite Seams at Garrison Dam.—Experimental grout studies (19) were conducted in July 1948 on the lignite seams in the Fort Union formation at Garrison Dam. The tests were made in the powerhouse area as considered

TABLE 1.—RECOMMENDED LIMITS

Mixture	Parts by Weight	Parts by Volume, in gallons
Portland cement	1	
Sand	6-20	
Emulsified asphalt per cubic foot of sand		0.1 to 1.0
Water per cubic foot of dry mix		4.5 to 8

the most suitable for subsequent excavation to observe and study the effectiveness of the grout pattern. To determine the feasibility of grouting the lignites, the following problems were explored:

1. Methods of grouting;
2. Grouting pressures;
3. Water-cement ratios;
4. The most effective and economical quantity of grout for each hole;
5. Spacing of holes; and
6. Size of holes.

The packer used for most of the tests seems to have been unusual. It was developed from soft rubber hose, had an outside diameter of 3-1/4 in. and an inside diameter of 2-1/4 in., and was cut in lengths of about 2-1/2 ft for these particular tests. The packer was assembled onto a packer coupling end. The upper coupling was movable to allow the packer to expand against the bore of the hole without exerting an undue strain on the packer material at the coupling. Metal straps were used to fasten the packer to the couplings. However, it was necessary to place tape under the straps to prevent cutting of the rubber by the straps. The expansion of the packer against the sides of the hole in the soft Fort Union formation was best obtained and maintained through the use of water instead of air as the expanding force.

The following pertinent conclusions were derived from the tests:

1. The 4- and 7.5-ft-thick lignite beds in the powerhouse area of the west abutment of Garrison Dam can readily be grouted.
2. These lignite beds can be grouted at one time or separately.
3. Either the "full-depth" or "stop" method of grouting was satisfactory.
4. Water-cement ratios ranging from an initial 1.5-to-1 to 0.7-to-1 appeared to be the most satisfactory in the test area.
5. The rubber packer performed satisfactorily with water pressure used as the expanding force.
6. A pressure differential of 25 psi on the packer was satisfactory.

The grouted lignites were examined in excavations made for the purpose of studying the joint patterns and the success of the grouting (20). The examinations indicated the following facts:

1. Both the 4- and 7.5-ft-thick lignites were readily grouted and it was assumed that other lignites in the same area could also be successfully grouted.
2. The stop method, instead of the full-depth method, should be used in grouting the lignites in this area because there was almost no grout in the 4-ft-thick bed where the full-depth method had been used.
3. The use of a packer made it possible to obtain more accurate information on the horizon to be grouted and to accomplish better washing of the bed during pressure-testing.
4. A spacing of 20 ft would prove satisfactory in obtaining an overlap grout pattern, provided the stop method of grouting is used.
5. Horizontal bedding planes or joints serve as grout channels. The pattern and continuity between the vertical and horizontal joints were excellent.
6. On the basis of the number of joints observed per unit area on the surface of the two lignites, the percentage of voids was computed as approximately 0.8. Prior to and during the testing program, it was assumed that the percentage of voids in the lignites was 0.5.
7. The relative ease with which these lignites were test-grouted indicated that high pressures and thin grout mixes would obviously be inapplicable and would result in an uneconomical use of grout by forcing it long distances on each side of the curtain.

Grout for Stabilizing Soft Soils, San Francisco Bay.—An investigation (27) concerned with the feasibility of mixing portland-cement grout in place with soft soils to form a stable mass was conducted by Intrusion-Prepakt, Inc., under the direction of the U. S. Naval Civil Engineering Research and Evaluation Laboratory. The research included both laboratory and field studies of methods of mixing with special emphasis on their applicability to naval construction, particularly in water-front foundation problems. In the laboratory tests, wooden boxes were filled with various soft soils, and various mixing heads were used in grouting the soils. When the soil in a box had been grouted and attained its set, the box sides were removed for examination and sampling of the soil.

Grouting materials used were standard portland cement type I, fly ash, a water-reducing, expanding grouting aid, uniform clean sand, and water. To provide a measure of the effectiveness of the grouting, the following control tests were performed on the grouted soil: (a) Strength (including compression

cubes and tensile briquets), (b) composition, and (c) miscellaneous others such as determination of soil properties. Grout-line pressure measurements were also made.

The field tests were performed in natural mud deposits at San Francisco Naval Shipyard, Hunters Point, on the west side of San Francisco Bay. The method consisted primarily of mixing grout into the soil by pumping while the soil was in place. This was accomplished by employing an auger or mixing blade with grout outlet ports in the lower end, and with hollow shaft. As the auger advanced into the soil, a controlled quantity of grout was pumped through the hollow shaft and distributed through the outlet ports. Thus, a column of grouted earth was created. A light core-drill apparatus, with a blade-type mixing head substituted for the usual bit, was used to accomplish the work.

It was concluded from the study that the process of mixing grout in place with soft soils to form structural elements (such as piles) is feasible within limits, but is more adaptable to repair jobs and small-scale construction than to large permanent works. By exercising proper control over mixing procedures, and grout type and quantity, and by making shallow trial units, a "pile" of satisfactorily uniform cross section can be produced to a depth of at least 40 ft in very soft soils. The depth is limited by the power of the equipment available and by the soil which must have very low shear strength and be saturated. Specific conclusions derived from the investigation were as follows:

1. The uniformity of grouting depended largely upon quantity of grout introduced, type of mixing head used, and the number of head revolutions per lineal foot. Usually the center portion of the unit was richer in grout than the outside portion and the grout did not permeate appreciably beyond the diameter of the mixing head.
2. The strength of the field mixes was erratic and could not be predicted accurately from laboratory mixes using the same quantities of the same materials.
3. It did not appear feasible to use less than 35% grout in the field mixes and the tensile strengths of the soil-grout mixes were very low.
4. The grouts containing fly ash and the grouting admixture offered advantages over neat-cement grouts from the standpoint of strength and pumpability.
5. The most suitable mixing head for placing soil grout units was found to be the individual blade type as contrasted with the continuous auger type. Advantages of this method of grouting are the low overhead clearance required for placing and the relatively light equipment used in the operation as compared with conventional pile-driving equipment.

Grout for Mixed-in-Place Piling.—E. R. Colle, M. ASCE (37) reported an investigation undertaken for the Department of Forests and Water Division of Flood Control, Commonwealth of Pennsylvania, to determine the feasibility of using piling formed by mixing grout in place by use of an auger with the soil, sand, and gravel on the river side of the concrete floodwall of the Lackawanna River at Scranton, Pennsylvania, to stabilize and protect the wall.

Six 20-in. diameter test pilings 16 ft into the underlying silt and sand were made in groups of three each so spaced as to overlap each other. The presence of boulders to a depth of 7 ft to 9 ft made it necessary to excavate the test area to that depth and backfill with riverbed material from near the site. In the first group of three the grout proportions were 3 bags of cement, 1 bag of

mineral filler, 1% chemical aid by weight of cement and 17 gal of water. In the second group of three the proportions were 2 cement to 1 filler plus 1% chemical aid and 15 gal of water. Piles 1 and 4 used 30% grout; piles 2 and 5 used 40% grout; and piles 3 and 6 used 50% grout.

After 28 days diamond cores were removed from the test piling drilled into the portion of each piling made from the grout mixed with the silt underlying the backfilled area. Strengths ranged from about 700 psi to 1650 psi, with 50% grout being required to produce the higher strengths. Although the desired compressive strengths of 2500 psi and 3000 psi were not obtained, it was the opinion of the reporting engineer that higher strength than that indicated could be realized by allowing for the overlapping of the piling and using larger diameter than the 4.8-in. cores tested. The consensus expressed in the Colle report was that a mixed-in-place pile wall was a practical application for insuring stability of the floodwall and preventing loss of foundation material from under the wall.

Grout for Mixed-in-Place Cutoff Wall.—Another investigation, reported by Colle (39), was made for the purpose of determining whether or not the mixed-in-place pile technique could be used to form an impervious core wall in proposed levee construction for flood control in Stroudsburg and East Stroudsburg, Pennsylvania.

Six laboratory mixtures gave promise of fulfilling the requirements of satisfactory cutoff. The parts by weight of the basic grout for three of the mixtures were: 1 cement:0.4 mineral filler:0.28 bentonite:1.78 water:0.0141 chemical aid. The parts by weight for the basic grout for the other three mixtures containing no bentonite were as follows: 1 cement:0.39 mineral filler:0.57 water:0.0139 chemical aid. Each grout mixture was then mixed with 30%, 40%, and 50% soil by volume from the test project site and small cylinders and beams were made for compressive and flexural strength, deflection, modulus of elasticity, permeability, and setting time.

Little difference was noted in the deflection at rupture between the specimens with and without bentonite. Permeability of specimens with both grouts approached that of conventional concrete. Strength of the bentonite-grout specimens (330 psi-500 psi at 28 days) was much less than the neat-grout specimens (3530 psi-4600 psi), but as flexibility and watertightness were considered more important than strength, it was decided to try the cheaper grout containing 20% bentonite, as well as the grout without bentonite, for the mixed-in-place test piles to form the impervious core wall. Five piles with 20% bentonite and eight without bentonite were made. Two piles apart from each other contained bentonite; one apart from the others was without bentonite. Three piles were made on 14-in. centers with bentonite, two on 14-in. centers without bentonite, and five on 12-in. centers without bentonite. All piles were 15 in. in diameter.

Difficulty was experienced in attempting to extract cores from the test piles because of the oversized pebbles in the piles and because of the tendency of the grout with bentonite to crumble. The strength of the cores without bentonite at 29 days was 1600 psi-1800 psi and with bentonite 150 psi-180 psi.

The recommendation of the engineer was to use grout with 20% bentonite and to excavate along the line of the cutoff to remove troublesome boulders and backfill prior to making the cutoff with material with top size not exceeding 4 in.

Use of Lumnite Cement.—Many uses and characteristics of calcium-aluminate cement which might prove of value in some phases of grouting are available (24). Although research is not noted therein (24), information is presented which might be found useful for special grouting conditions, and therefore is described here. Aluminous cement sets and hardens when mixed with water and can be subjected to the full load for which it is designed in less than 24 hr after placing. Some of the pertinent special uses of aluminous cement are as follows:

1. For grouting leaks where quick-hardening properties are highly advantageous, such as in work around dams, mines, and tunnels;
2. in industrial grouting, such as setting bed plates for heavy machinery and equipment; and
3. in quick-setting gunite mixtures.

When portland cement is mixed with aluminous cement, the set is greatly accelerated, frequently causing flash set. Such mixtures are not recommended for ordinary construction work, but when placed with a cement gun the aluminous cement-portland cement mixtures may solve difficult construction problems. In the cement gun, the water joins the cement at the nozzle of the gun. Thus, flash-setting mixtures may be used because the pneumatically-placed mortar is in place before setting can occur. This method is convenient for special conditions, such as shutting off water, sealing caissons, and stopping seepage in rock seams.

Other Reviews of Grouting.—A thorough review of equipment, materials, and procedures used in foundation treatment was presented by A. W. Simonds, Fred H. Lippold, Fellows, ASCE, and R. E. Keim (25) in 1950. The information they presented is based, for the most part, on experiences gained in grouting the foundations of twenty or more large dams. It is concerned with the application of neat-cement grout, although the use of other foundation grouting materials, such as asphalt, bentonite, and clay, is also described.

In 1955, Bruce E. Clark, Aff. ASCE (30) assembled the fundamental principles involved in grout penetration. Although no new research of cement grouting is described, many conclusions are listed.

Use of Injection Devices and Ultrafine Cement at Shaver Dam.—During the years 1945, 1947, and 1949, the Southern California Edison Co., Los Angeles, Calif., conducted experimental programs in grouting leaking construction joints at the company's Shaver Dam. A specially developed cement and equipment furnished by the Portland Cement Association (PCA) were used in the work. Grouting successfully stopped some of the leakage but water was still emitted by some of the very fine cracks. Three methods of grouting, none of which was entirely successful, were tried in the experimental program.

1. The hand-thrust method was tried in which gun seats anchored astride the injection holes in the leaking construction joints, and a rubber-tipped grout injection gun were used. In this method the heavy pressure required to obtain penetration made it difficult to maintain contact between the gun seats and the guns.
2. Holes were drilled in the construction joints and expansion shields were placed in the holes. A specially made pipe connected the hole with the pump. Fines produced by the drilling could not be removed completely; consequently, the cracks were bridged, causing grout refusal.

3. A new injection device was used which proved to be the most successful of the three methods of grouting tried. The device consisted of a rubber-gasket-sealed plate with a hole in the center over which a 3/8-in. pipe nipple was welded. The plate was clamped over the injection hole by bolts inserted through the plate into lead expansion shields placed in the dam face.

Grout used in this program was prepared from 0-40 micron fraction of high-early-strength portland cement. It was found that a grout mix of 4.0 to 5.0 water-cement ratio was required to penetrate some joints.

The High-Speed, Dual-Drum, Circulating-Type Mixer.—In February 1955, a report (28) on the use of a high-speed, dual-drum, circulating type grout mixer at Folsom Dam was issued by the Sacramento District, Corps of Engineers. The machine is intended principally for mixing mortar grout and is claimed to produce a more stable, fluid grout than the regular-speed paddle-type mixer. Comparative tests made with this mixer and a standard paddle-type mixer indicated that mixes of substantially equal stability could be produced by conventional mixers and methods if the normal mixing time is increased sufficiently.

The advantages of the high-speed mixer were summarized as follows:

1. Better mixing of the grout and greater freedom from lumps are achieved;
2. Mixer can be at ground level. Cement can be taken directly from back of truck with less lifting and handling;
3. Discharge can be pumped to some distance from and a little above the mixer by its pump;
4. The apparatus is highly mobile, compact, and flexible;
5. It is very easy to clean; and
6. The mixer mixes small batches, and is therefore better adapted to sensitive control of water-cement ratio.

The following disadvantages of the high-speed mixer were reported:

1. Heats grout or water left in running mixer;
2. Is more expensive to buy and maintain than conventional mixers; and
3. Requires a little more care in operation than a conventional mixer, particularly in fast operation.

LABORATORY RESEARCH IN THE UNITED STATES

Bureau of Reclamation.—The USBR has done a considerable amount of investigation in the development and use of suitable instrumentation to measure fluidity, flowability, or consistency of grouts. Instruments that have been tried are the Standard Oil Company's oil well testing apparatus, a piano-wire torque meter, an inclined pipe flow meter, shot-cell flow meter, a glass tube and funnel viscosimeter, and the Stormer viscosimeter. The fluidity of some grouts can be measured well in one type of apparatus but not in another. The consistency of sanded grouts, for example, cannot be readily judged by observations with some viscosimeters, and the torque meter is not sufficiently sensitive to measure slight changes in viscosity of very thin neat grouts.

The oil well testing apparatus measured viscosity and mixing time by use of a complicated procedure not necessary for ordinary grouting.

In attempting to obtain equipment suitable for measuring the consistency of grout, the USBR built a device, called a torque meter, based on design and

plans from Raymond E. Davis (13). The torque meter was a device wherein a turntable on which a pan of grout 2 in. deep and 7 in. in diameter revolved. A pendulum weight in form of a disc weighting about 23 lb with two crossed 1/8-in. round rods attached to the underneath was suspended from a 35-1/2-in. length of No. 18 music wire above the revolving pan. The disc weight was graduated from 0° to 360°. Provision was made for immersing the crossed rods attached to the disc in the revolving pan of grout. The moving grout imparted torque to the suspended assembly. The thicker the consistency the greater the torque reading.

Tests to develop further equipment to measure flowability were made (16) for which the length of time required to discharge a fixed volume under low head (16 in.) through a 7-ft length of 1/2-in. pipe inclined upward at 32° was observed. This approach was not too successful and led to studies to develop the shot cell and the glass tube viscosimeter (15).

In development of the shot cell it was desired to obtain apparatus sensitive enough to detect small changes in the flowability or ability of thin grout to penetrate restricted openings or penetrate a bed of granular materials. The cell consisted of a transparent plastic tube containing a known weight of steel shot. The length of time required for a known quantity of grout to pass through the cell at a carefully controlled constant pressure provided a quantitative measurement of the flowability.

Tests of grout through the cell and through a viscosimeter made of small diameter glass tubing with a constricted area caused by drawing in a laboratory burner flame indicated that flowability and viscosity are closely related. Both were found to be suitable to indicate small differences in the flow characteristics of cement-water slurries. The Stormer viscosimeter, a piece of commercial equipment, was found to be very suitable for measuring the viscosity (consistency) of neat grouts (33).

The USBR has conducted considerable research into the effects on the properties of grouts of adding organic materials and mineral fillers to grouts. The effects of passing cement through a colloid mill were observed.

The oil-well cement used in grouting at the Boulder Canyon Project in 1943 had desirably long setting time with good early and ultimate strength (8). Its cost, however, was $1.00 per bbl more than that of modified (type II) cement from the same mill.

It was felt that the addition of the correct admixture to the modified cement might impart the desirable properties of the oil-well cement at nominal cost. An investigation was made of a compound containing 87-1/2% calcium lignosulfonate and 12-1/2% triethanolamine. An oil-well cement was used as a control and the additive was used experimentally with modified cement from the same mill as the oil-well cement. The effect of the additive on slurry made with high-early-strength cement was also determined.

It was found that by use of 0.10% to 0.16% by weight of the cement the desirable properties of the oil-well cement were imparted to the modified cement. The admixture was prepared as a solution and added as the grout was mixed. The added cost for the admixture was about 7¢ per bbl. Viscosity was decreased, setting time increased, and strength unimpaired. The material was successfully used in the field. In contrast, with high-early the admixture caused premature stiffening and increased viscosity.

A rather extensive series of tests (13) involving small batches of grout were made and tested under several conditions for effect on consistency. Consistency was measured by use of the torque meter.

Two mixers were used. One was a small drink mixer; the other was similar to it but more vigorous in action. The effects of mixing time; of additions of sand (all of which was finer than the No. 30 sieve), fly ash, and several chemical admixtures; of special so-called low-solubility cement; and of temperature were observed. Voluminous tables of data were derived.

Briefly, it was found that fluidity increased with mixing time. Maximum fluidity was obtained after 10 min of mixing with the less vigorous mixer and in 5 min with the more vigorous mixer. Increasing water content decreased viscosity. The addition of sand without addition of water decreased fluidity. Increase in temperature decreased fluidity for grouts with low water (cement + fly ash) ratios. However, at ratios of 0.65 or more by weight consistency was little affected. The effect of fly ash was to markedly decrease consistency at a constant water to (cement + fly ash) ratio as the amount of fly ash to cement increased. Initially of the same fluidity, after one hour of mixing, grout with 80% fly ash, 20% cement was about twice as fluid as grout with 100% cement. The admixtures containing lignin compounds appeared to decrease consistency. The "low-solubility" cement was supposed to behave in grout without fly ash as other cements would with fly ash. At equal water cements, ordinary cement plus fly ash provided grout of lower viscosity than did the special cement.

Another investigation (9),(12) beyond that previously described (8) was made to find an admixture that, when added to modified (type II) cement, would retard its set without impairing its strength thus making it behave similarly to a good oil-well cement. Such materials would be of value in tempering grout to be used in areas where hot water is encountered and in circumstances where it might be desirable to prolong the setting time.

Twenty-two admixture materials or combinations of materials in several different proportions were tested for their effect on nominal consistency and setting time of paste, strength of mortar cubes, and viscosity of cement slurry containing 40% water.

The most effective and economical materials for the purpose of retardation without loss of strength were lignin compounds. Used judiciously they appeared to impart to modified cement the qualities and characteristics of a finely ground oil-well cement. Set was definitely prolonged, viscosity decreased, and they retained their retarding action under the conditions of test up to 140 F.

Further tests were made (11) by the USBR to determine the effects of adding to grouting slurry admixtures composed of (1) calcium lignosulfonate plus triethanolamine and (2) calcium lignosulfonate alone, to (a) one normal cement (type I), (b) ten modified cements (type II), (c) five high-early-strength cements (type III) and (d) one low-heat cement (type IV).

It was found that most of the modified cements responded very favorably to the addition of 0.16% to 0.30% of the lignosulfonate-triethanolamine admixture. The low-heat cement responded favorably. The normal cement became flash setting with the combination admixture, but behaved well with calcium lignosulfonate alone. Only two of the five high-early-strength cements responded favorably to the combination admixture. All five responded favorably to calcium lignosulfonate alone, but not to as great an extent as the modified cements or the normal cement.

The investigation indicated that the combination of triethanolamine and calcium lignosulfonate should be tried in the laboratory to observe its effect on the cement selected for use in the field before actual trial on the job

In order to determine if passage of cement grout through a special mill (10) would cause appreciable reduction in grain size of the cement thus resulting in a material which would better remain in suspension, and exhibit better flowability (penetration would be improved if grain size were reduced), tests were made with a "Charlotte" "Colloid" mill.

The mill was borrowed from the Colorado School of Mines and tests were made with clearances between rotor and stator of 0.0030 in. to 0.0150 in. Slurries of ethanol and cement were used because in correct proportions they would produce slurries similar in characteristics to water and cement, but there would be no reaction between the ethanol and the cement. Cement suspensions of ethanol to cement (milliliters per gram) of 0.40 and 0.60 were passed through the mill at various settings, dried and the change in fineness of the cement determined by a turbidimeter.

It was found that the maximum change in fineness with passage through the mill was about 8% increase in specific surface. Actual grinding was negligible because the closest setting was 0.030 in. (76 microns) which would only grind material considerably larger than the 325-mesh sieve (44 microns). Most of the increase in fineness must have been due to dispersion. Subsequent passes of the slurry through the mill after the first pass were much less effective than the first pass.

Flowability was greatly improved and viscosity greatly decreased by passage through the mill. Curiously, the rate at which the solids settled from a column of slurry in a test tube was appreciably increased.

The effects of bentonite on the properties of neat grout were investigated (33). A series of thirty-five mixtures containing 2% bentonite by weight of cement was made. In some of the mixtures the bentonite was added dry to the cement before mixing with water; in others it was premixed with the water before the cement was added. The effects on consistency, settlement, strength, and pumpability were determined.

It was found that bentonite thickened the grout at the same water content, but the thickening was not considered objectionable at water-cement ratios of 1:1 to 3:1. Bentonite markedly decreased the amount the solids in the grout settled on standing. At equal consistencies bentonite produced a marked reduction in strength. The maximum thickening effect was obtained when the bentonite and water were premixed. The lime and alkalies leached from the cement were believed to inhibit complete swelling of the bentonite.

Data are reported on 104 mixtures (34) on which tests were made to determine the effects on fluidity, strength, and settlement, of using retarders, dispersing agents, and fillers such as fly ash, pumicite, and bentonite, and various combinations of admixtures and fillers.

Most of the grouts had a water to solids ratio of 1.57 by absolute volume. The mixtures were pumped through 250 ft of 1/2-in. pipe with gages along the pipe and the time required to discharge 1/2 cu ft of grout from the end of the line was observed. The same pressure was maintained at the pump for all grouts.

It was observed that the values for the coefficient of velocity obtained from the pumpability data did not agree with those obtained with a standard viscosimeter. It is believed that slurries do not behave as true fluids, but rather as true plastics. Harold E. Babbit and David H. Caldwell treat this phenomenon

fully.[3] A study of their work leads to the belief that the Stormer viscosimeter can be used to accurately predict pumpability of neat grouts.

The investigation revealed that thick grouts were made less viscous by some dispersing agents and by the substitution of up to 30% fly ash for cement without detrimental effect on strength. The substitution of pumicite, diatomaceous earth and bentonite for cement increased the thickness and decreased the pumpability of the grout. The dispersing agents containing accelerators while causing increased rate of strength gain caused rapid thickening of grouts with some cements.

In discussing a paper by A. W. Simonds (17) on contraction-joint grouting of large dams, Simonds and Walter H. Price, F. ASCE (18) described USBR tests which were made to determine the minimum joint opening that can be grouted and the efficiency of grouted joints in transmitting stresses. Although the tests were made to obtain information needed in grouting contraction joints, the results are also useful in the grouting of small openings in foundations. The tests showed that it is possible to grout effectively a joint having a width of 0.005 in. using cement screened through a 200-mesh sieve. Complete coverage of the joint area was not always obtained with this small opening, and pressures as high as 500 psi were required to force 0.75 water-cement ratio grout through the joint. It was indicated that an opening of about 0.008-in. width is the minimum that can be effectively grouted with cement grout passing a 200-mesh sieve.

Waterways Experiment Station.

Grouting of Foundation Sands.—The results of a study performed by the Soils Division, Waterways Experiment Station, to investigate the feasibility of grouting foundation sands using modifications of existing grouting procedures, with particular reference to cement grouts, and to consider the potentialities of grouts with new materials, were reported (29) in June 1955. The investigation consisted of two parts:

1. A review of published information on injection grouting theory and application, and factors affecting the success of grouting.
2. A laboratory investigation program, guided by these findings, to determine the potentialities of commercial type III (high-early-strength) portland cement, scalped type III portland cement, slag cement, ground slag, and calcium acrylate grouts in grouting foundation sands; to explore the possibilities of improving these grouting agents through modification of the injection process or technique; and to develop a tentative criterion for predicting the success or lack of success that will attend the grouting of a particular sand with a certain grout.

Several references were listed in the report but only a few were discussed fully in the literature review. The more relevant and important points in the discussions are as follows:

1. C. F. Kollbrunner and C. Blatter (7) stated that injection of cement suspensions is possible only at a pore-size diameter larger than 0.1 mm, and the lower limit of grain size for sands subject to grouting with cement

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[3] "Laminar Flow of Sludges in Pipe with Special Reference to Sewage Sludge," by Harold E. Babbit and David H. Coldwell, Univ. of Ill. Experiment Sta., Bulletin Series No. 319, November, 1939.

is 0.8 mm. The required coefficient of permeability for successful cement grouting of sand is about 1000 x 10^{-4} cm per sec. Kollbrunner also reported that an injection can be successful only if the pressure increases as the soil pores become filled to capacity with injection material, and if the deposited materials are compressed by filtering off the liquid carrier. For this reason, a pump should be employed instead of air pressure in injection work and when refusal is reached, the pressure should be maintained for 40 to 50 min.

2. A. Machis, M. ASCE (21) experimented with cement slurries and concluded that no amount of pressure will make a cement slurry pass through sands with grains finer than 0.59 mm. However, penetrations in the order of 3- or 4-grain diameters were obtained in sands with a grain size of 0.59 mm. Air pressure was used to perform the injection tests and best results were obtained when the pressure was applied rapidly.

3. Karl Terzaghi, Hon. M. ASCE (5) reported that fissures with widths less than about 0.1 mm cannot be grouted with portland-cement grouts. If the effective grain size (D_{10}) of a compacted sand is smaller than 1.4 mm or that of a loose sand is smaller than 0.5 mm, the grout merely displaces the material by lateral compression and the result of the grouting operation consists of compact, tree-shaped bodies of solidified cement separated from each other by layers of sand with unimpaired permeability.

Laboratory tests were conducted to determine the minimum grain size of sand that could be successfully grouted with the potential grouts under consideration, and to collect data that might aid in the determination of the groutability of sands and in the selection of suitable grouts. In order to effectively evaluate the materials in both phases of testing, the following laboratory steps were carried out, for each specimen in the order given: (1) permeability determination before grouting, (2) grout injection, (3) moist curing, (4) permeability determination after grouting, and (5) unconfined compression test (when practicable). Five basic grout mixtures, in which the cement, sand (grain size) water-cement ratio, and admixture were varied, were tested. The following pertinent conclusions were developed from the test results:

TABLE 2

Grout	Sand Grain Size D_{10}, mm	Sand-Grout Ratio D_{15} Sand/D_{85} Grout
Scalped type III portland cement ($< 30\mu$)	0.29	18
Commercial type III portland cement	0.59	24
Slag cement	0.67	19
Ground slag	0.31	21

1. Each of the grouting materials presented in Table 2 will grout sands with grain size (D_{10}) and sand-grout ratios as indicated.

2. The anticipated low cost of producing cement for field grouting makes scalped type III portland cement the most promising potential grouting agent studied.

3. A water-cement ratio of 2:1 is near optimum and may be expected to produce a grout of maximum penetration and stability.

4. Calcium lignosulfonate did not have a beneficial effect upon cement grouts.

5. Grouting with the cements tested is not possible when the sand-grout ratio $\left(D_{15} \text{ size of sand}/D_{85} \text{ size of grout}\right)$ is below 11.

6. Grouting is possible when the sand-grout ratio is more than 24. Although additional tests will be required to more closely define the sand-grout ratio range of groutable material, it appears that a sand-grout ratio of 19 may represent the limiting sand-grout ratio.

Pumpability of Sanded Grouts.—T. B. Kennedy and J. M. Polatty (31), in 1955, reported the results of the first phase of a comprehensive study of sanded grouts. In this phase, pumpability tests were made of cement grouts that included chemicals and mineral fines. Laboratory- and field-type mixing and pumping equipment were used in determining the amount of natural sand passing a No. 16 sieve that can be added with certain chemical admixtures and finely ground mineral fillers to portland-cement grout without injuring its pumpability. This information was sought because it is frequently desirable, from an economic as well as a structural standpoint, to use a cheaper material than neat cement in grouting foundation cavities.

The pumping tests were made with cement, sand, and water in combination with various admixtures. The admixtures tested were commercial fluidifying admixture, diatomite, methocellulose, bentonite, calcium chloride, calcium lignosulfonate, and sucrose. Other tests to determine the effects of prolonged pumping and mixing on sand-carrying capacity of grout containing an inert material substituted for cement were made with fly ash. Laboratory tests were made on small batches of cement-sand grout to determine the approximate water requirements and bleeding that might be expected with various ratios of cement-to-sand at a relatively constant consistency. The consistency of the grout was measured by means of a torque consistency meter and a Stormer viscosimeter. Compressive strength and time of set were determined on grout that had the longest pumpability record for each combination.

General conclusions based on the results of the tests were as follows:

1. It is feasible to pump sanded grouts.

2. Two parts of ordinary concrete sand after passing through the No. 16 sieve to one part of cement can be pumped without the aid of admixture at normal temperature.

3. Bleeding did not correlate with pumpability.

4. Pumping technique was of utmost importance. Sudden application of pressure after interruption in pumping and before good flow was re-established resulted in obstruction in the lines.

5. The age of the grout (the length of time it had been mixed and pumped) had an effect on its pumpability; the older the grout, the more sand it could carry and still remain pumpable. Visually it appeared that the older the grout, the more unctuous and homogeneous it became and the better it pumped. However, the grout stiffened with age and required addition of water to maintain original consistency.

6. The temperature of grouts containing portland cement apparently influenced their sand-carrying capacity. The higher the temperature, the more sand they carried.

7. The grout fluidifying admixture and methocellulose increased the sand-carrying capacity of the grouts to a small extent, diatomite to a greater extent, and bentonite to a very great extent; however, the use of more than 10% bentonite permitted incorporation of such a large amount of sand that practically no strength resulted.

8. The use of calcium chloride had little or no effect on the sand-carrying capacity of the grouts. Calcium chloride accelerated the set and had a beneficial effect on the early strength.

The influence of sand grading and addition of mineral fines on the pumpability of cement grout was investigated during the second phase of the sanded grout investigation program, and the results were reported (32) by J. M. Polatty, F. ASCE in October 1955. The second phase also included two series of tests to determine the maximum amount of sand that could be pumped in a sand-cement grout using sands of six different gradations. In the first test series, five percentages of sand, from 0% to 25%, passing the No. 100 sieve were studied. In the second series, a sand deficient in material passing the No. 100 sieve was used and the effect of adding increments of finely divided mineral fillers such as diatomite, fly ash, pumicite, and loess, was determined. Mineral fillers were added in increments that varied up to an amount equaling the weight of the cement.

Test results indicated that an increase in the amount of sand passing the No. 100 sieve from 0% to 25% permitted the ratio of sand to cement in pumpable grout to be increased from 2:1 to 3:1 by weight. However, the water-cement ratio with the added fines increased from 0.63 to 0.87 by weight, while the 28-day compressive strength dropped from 3505 psi to 2120 psi.

The following conclusions were developed from the results of tests performed during this phase of the investigation.

1. Ordinary concrete sand scalped over a No. 16 sieve can be successfully used for grouting purposes.

2. When the sand is deficient in material passing the No. 100 sieve (nominal 0%), grouts with a ratio of one part portland cement to two parts of sand can be successfully pumped without the use of an admixture.

3. When the sand is deficient in material passing the No. 100 sieve, the addition of finely divided mineral admixtures increases the sand-carrying capacity of the grout.

4. Diatomite, the finest of the admixtures tested and lowest in specific gravity, was the most effective in increasing the amount of sand that can be used in a grout; loess, the coarsest and highest in specific gravity, was the least effective.

5. No correlation was apparent between the bleeding and the pumpability. The bleeding increased with the addition of fine sand and was reduced with the addition of diatomite. The addition of fly ash, pumicite, and loess did not appreciably affect the bleeding.

6. In each of the series of mixtures, the water unit requirement increased with addition of fine material to maintain a constant consistency and the relative cement content actually decreased, resulting in a decrease in the compressive strength of the grouts.

A third phase of the investigation to determine the influence of grading and specific gravity of manufactured sands on pumpability of grouts, was reported

by Polatty (36) in February 1957. In addition to determining the maximum amount of manufactured limestone and traprock sands that could be incorporated in a pumpable grout, an attempt was made to evaluate the mixing properties of a high-speed mixer.

The grouts used in the pumping tests of this phase consisted of the following combinations of materials:

1. Cement, water, and limestone sands of three gradings, nominal 0%, 10%, and 25% finer than the No. 100 sieve size.
2. Cement, water, and limestone sand, none of which was finer than the No. 100 sieve size, with the addition of approximately 11%, 25%, 43%, 67%, and 100% of both fly ash and loess based on the weight of the cement.
3. The same combination of materials listed in 1 and 2 except that traprock sand was substituted for the limestone sand.

The tests of grout mixtures containing limestone sand showed that (a) an increase in the amount of sand finer than the No. 100 sieve size resulted in a large increase in the sand-carrying capacity of the mixtures, and (b) when fly ash or loess were added to mixtures containing sand with basically no material finer than the No. 100 sieve size, the sand-carrying capacity of the grouts was increased.

The tests of grout mixtures containing traprock sand revealed that (a) an increase in the percentage of traprock sand finer than the No. 100 sieve size resulted in an increase in the total amount of sand that could be used in a pumpable grout, and (b) the addition of fly ash to the traprock sand with essentially no material finer than the No. 100 sieve size afforded results similar to those obtained in the limestone sand-fly ash tests.

Upon examination of the pumping test results, it was noted that the increase in sand-carrying capacity of grouts incorporating traprock was relatively small compared to that of the grouts made with limestone sand. In addition, traprock finer than the No. 100 sieve was not as effective in promoting pumpability as the limestone sand. It was found possible to pump 1.75 parts of both limestone and traprock sand to 1.0 part of cement by weight when the sand contained no material finer than the No. 100 sieve size, and 7 parts of limestone sand or 2.25 parts of traprock sand to 1.0 part cement when 25% of either sand was finer than the No. 100 sieve size.

The following conclusions were derived from a study of the test data:

1. Sands manufactured from limestone and traprock can be successfully used in sanded grouts;
2. The specific gravity of the sands within the range tested had little or no effect on the pumping characteristics of the grout;
3. Both fly ash and loess were effective in promoting pumpability when used with sands deficient in material finer than the No. 100 sieve size;
4. The limestone fines used in these tests, although having somewhat lower specific surface values than fly ash or loess, were pound for pound more efficient in promoting pumpability than the fly ash or loess;
5. Traprock fines, being relatively coarse, were not as efficient as fly ash or loess and were much less efficient than limestone in increasing sand-carrying capacity;
6. Setting time of the grout appeared to be lengthened slightly by addition of fly ash and loess;

7. Compressive strength varied with water content but that of the mixtures containing fly ash was slightly higher at 28 days;

8. Bleeding was minor for all the mixtures; and

9. The results of tests for the evaluation of the high-speed mixer were inconclusive.

A fourth phase of an investigation of sanded grouts was reported by J. M. Polatty (38) in October 1958. The pumpability of grout with limestone sand was investigated when additional fine material in the form of fly ash or diatomite was added to the mixture. The pumpability of grouts prepared in the high-speed mixer was compared with grouts prepared in a paddle mixer.

It was found that the addition of fly ash or diatomite had about the same effect as the addition of cement or other equally fine material. It was found possible to pump 3 parts of sand to 1 part of cement if the grout also contained 11% fly ash by weight of the cement. With the same amount of diatomite 4.5 parts of sand to 1 part of cement could be pumped. With grout containing 1 part of cement and 1 part of fly ash, 7 parts of sand could be pumped. With the same amount of diatomite 12 parts of sand could be pumped.

The pumpability and other characteristics of grouts of similar ingredients and proportions were similar whether mixed by the high-speed or paddle mixer.

Pressure Grouting of Fine Fissures.—In October 1956, H. K. Cook and T. B. Kennedy reported (35) an investigation involving several tests and procedures for pressure grouting fine fissures. The purpose of the program was to obtain information on the degree to which the penetration of fine fissures by grout is influenced by surface texture of the seam, pumping pressure, water-cement ratio, chemical fluidifiers, and finely divided mineral additives. It was also desired to determine the effect of these factors on the quality of the hardened grout films.

The study was conducted in three stages:

1. The first stage furnished data on the lowest water-cement ratio grout that could be pumped through fissures of 0.01-, 0.02-, and 0.03-in. thickness at 100-psi pressure, using standard field equipment and methods and the following grout mixtures: (a) Neat cement, (b) cement plus fly ash, (c) cement plus fly ash plus a grouting aid. A study of consistency, bleeding characteristics, and setting times of the various grouts was also made. After the grouting tests were completed, chemical and petrographic examinations of bleed water and solid residues from some of the high water-cement ratio grouts, and further bleeding tests were made.

2. The second stage provided information on grout penetration obtained at pumping pressures of 25 psi and 50 psi using the three grout mixtures of the first stage and an additional mixture containing cement plus the grouting aid. Tests were also made of neat-cement grout plus calcium lignosulfonate pumped at 25 psi, 50 psi, and 100 psi, and of neat-cement grout plus a grouting aid pumped at 100 psi.

3. The third stage consisted of pumping tests (through a 0.03-in. crack) at 50 psi of the following seven grouts: (a) Neat cement, (b) cement plus fly ash, (c) cement plus calcium lignosulfonate, (d) cement plus fly ash plus calcium lignosulfonate, (e) cement plus slag, (f) cement plus pumicite, and (g) cement plus calcined opaline shale. Stage three also included tests for consistency, bleeding, setting time, quality, and the solubility in distilled water of hardened grout films.

Ninety-seven pumping tests were conducted. The following conclusions were based on the data derived from the tests:

1. The surface texture of a fissure has a distinct influence on the thickness of grout that can be used to fill it. The smoother the surface, the lower the water-cement ratio of grout that will penetrate the fissure.
2. The maximum grain size of the solids in the grout determines the minimum width of fissure that can be grouted. The ratio of fissure width to grain size should probably be three or more.
3. The use of such materials as the grouting aid or calcium lignosulfonate increases the fluidity of grouts to some extent, thereby promoting penetration of a given crack with grouts of a slightly lower water-cement ratio than could be used successfully without them. Bleeding can be reduced somewhat and setting time increased by use of these materials.
4. It is impracticable to squeeze the excess water from a thin grout so as to leave a dense, hard filler in the cavity, at the pressures used in this program.
5. Bleeding largely prevents bonding of the grouting material to the upper surface of the fissure.
6. The use of finely ground mineral admixtures such as granulated blast-furnace slag, pumicite, and opaline shale can reduce the bleeding of a grout and greatly improve the continuity and appearance of the hardened grout film.
7. The solubility of a grout film is influenced by the water-cement ratio and composition of the film, and length of curing. Certain mineral admixtures such as granulated blast-furnace slag and calcined opaline shale can be used to reduce the amount of leaching that a grout film undergoes.
8. A straight-line pressure gradient occurs along a fissure being grouted only if the fissure is of sufficient width.
9. Neat-cement grout with a water-cement ratio of 0.43 penetrated the 0.03 in. wide fissure at 25 psi, but it was necessary to increase the water-cement ratio to 2.67 before neat grout would penetrate the 0.01 in. wide fissure at the same pressure.

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SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

SYMPOSIUM ON GROUTING: INVESTIGATION OF SAND-CEMENT GROUTS

By James M. Polatty,[1] F. ASCE

FOREWORD

This paper is one of several papers that constitute the Symposium on Grouting which occupied two sessions of the June 1960 meeting of ASCE in Reno, Nevada. Credit for organizing this symposium program goes to a Special Grouting Symposium Committee consisting of Stanley J. Johnson, F. ASCE, Chairman, T. William Lambe, F. ASCE, and W. K. Seaman, F. ASCE.

It is believed that these several papers constitute a significant contribution to engineering practice in the art and science of stabilizing foundation materials by grouting. This is a field in which there appears to be increased interest among foundation engineers and one which is expected to become more important as time goes on and the need to build structures on difficult soil conditions increases.

The papers of the symposium are concerned with cement, clay, and chemical grouting of seams, cavities, and granular materials. The papers illustrate the increased use of chemicals for grouting purposes and also their use with more conventional grouting materials such as clay and cement; grouting of overburdened materials which is rapidly becoming a more definite art and science and which, to some degree at least, is susceptible to laboratory studies; the grouting of a cut-off in granular materials to prevent underseepage beneath a major hydroelectric project (perhaps the first application of this type

Note.—Discussion open until September, 1961. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

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of grouting in the United States); and the grouting of subsurface voids and solution channels in limestone areas subject to large subsidence.

Organized in 1952, the Grouting Committee with Task Committees on Cement, Clay, and Chemical Grouting has sponsored groups of papers or symposia occupying the sessions of meetings of the Society in 1954 and 1956, as well as 1960. Papers on grouting which have been published in the Journal of the Soil Mechanics and Foundations Division are as follows: Nos. 1145, 1204, and 1544 to 1552 inclusive. The report of the Task Committee on Chemical Grouting appears as Paper 1426 in the November 1957 Journal. A draft of a report on clay grouting has been prepared by the Task Committee on Clay Grouting, and the report on cement grouting prepared by the Task Committee on Cement Grouting is substantially in its final form.

The Administrative Committee on Grouting takes this opportunity of expressing its appreciation to the task committees for the excellent manner in which they have carried out their assignments and to the individuals who, as writers of papers, have made significant contributions covering the practices of grouting in various fields.

Raymond E. Davis, F. ASCE, Chmn., Admin. Committee on Grouting, Prof. of Civ. Engrg., Emeritus, Univ. of California, Berkeley, Calif.

SYNOPSIS

The purpose of the investigation was to determine the maximum amount of different types and gradations of sand that could be used in a portland-cement grout without injuring its pumpability. In addition, studies were made of the effect of various admixtures on sand pumpability, using sands that either were deficient in or contained ample amounts of material passing a No. 100 sieve.

INTRODUCTION

The U. S. Army Engineer Waterways Experiment Station under the authorization of the Office, Chief of Engineers, and in connection with the Corps of Engineers Civil Works Investigation Programs, has conducted a number of studies on the pumpability of sanded grouts.

Due to the extensive use of grouting on Corps projects, it was felt that such a program could furnish much valuable information for both design and con-

struction personnel, and could result in methods, procedures, and the usage of materials that would effect considerable cost savings and insure better results.

PURPOSE OF INVESTIGATION

The purpose of the investigation was, in general, to determine the maximum amount of sand that could be used in a portland-cement grout without injuring its pumpability. In order to widen the scope of the investigation, different gradations and types of sands were test-pumped. In addition, the effect of various admixtures on sand pumpability, using sands that either were deficient in or contained ample amounts of materials passing the No. 100 sieve, were studied.

MATERIALS

A standard Type II cement complying with Federal Specification SS-C-192b was used in all of the test mixtures.

Three types of sand were used in the test mixtures:

1. A well-rounded natural siliceous sand from Mississippi with a 2.6 specific gravity;
2. A manufactured limestone sand from Tennessee with a 2.7 specific gravity, and
3. a diabase traprock manufactured sand with a 2.9 specific gravity.

The gradation given in The Office of the Chief Engineer's (OCE) standard guide specification for concrete sand was used as a starting gradation for the sands. However, this specification allows an average of 35% to be retained on the No. 16 sieve. By removing (by scalping) the material retained on this sieve, the resulting average gradation is the "10% passing No. 100 sieve" as shown in Fig. 1. In addition to the gradation, sufficient material passing the No. 100 sieve was either added or removed to produce sands with gradations of 0%, 5%, 15%, and 20% passing the No. 100 sieve for the natural sand and 0% and 15% for the limestone and traprock sands. The gradations of the 0% and 25% passing the No. 100 sieve are shown in Fig. 1.

Five admixtures used in the investigation and their sources are as follows: diatomite, uncalcined, California; fly ash, Illinois; pumicite, California; loess, Mississippi; bentonite, Wyoming. Physical data for the materials used as mineral fines are given in Table 1.

EQUIPMENT

The mixing, pumping, and circulating system consisted of a standard laboratory-type paddle mixer of approximately 37 gal capacity, and an air-driven, simplex 2-1/2- by 3-1/4- by 5-in. grout pump. The mixtures were test-pumped through 200 ft of 3/4-in.-ID rubber hose which, after leaving the grout pump, rose vertically for 13 ft, made a 5-in.-radius turn, and returned to the pump elevation. The remainder of the hose was coiled in a 5-ft-minimum radius on

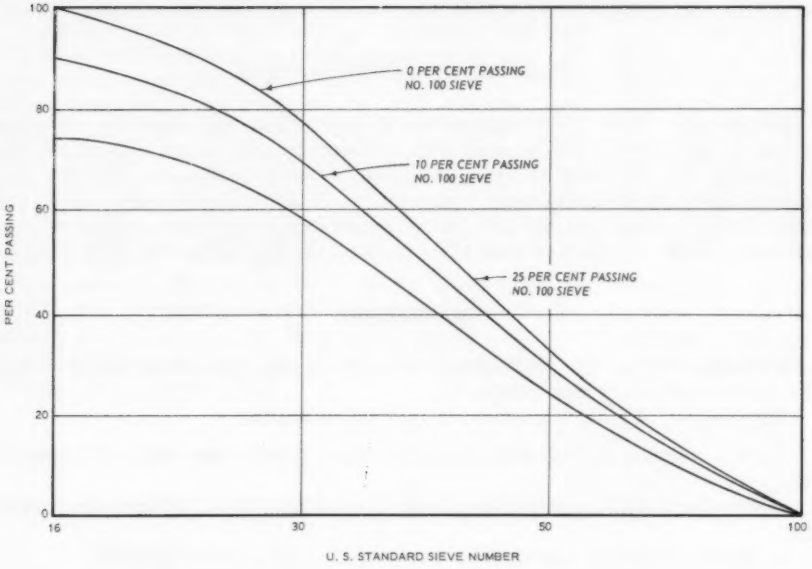

FIG. 1.—THREE OF THE FIVE SAND GRADATIONS USED IN THE INVESTIGATION OF SAND CEMENT GROUTS

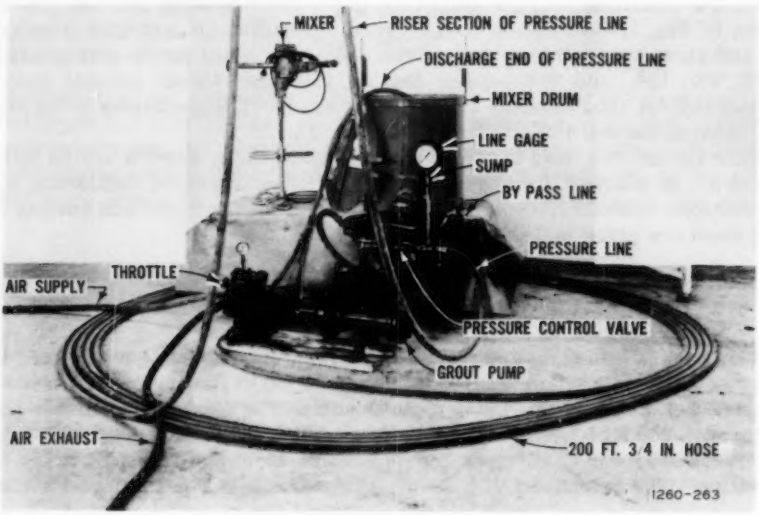

FIG. 2.—GROUT TEST PUMPING EQUIPMENT

the floor. The 200 ft of 3/4-in. hose consisted of four 50-ft sections, each section connected with a metal connector having a 5/8-in.-ID opening. Fig. 2 shows the test pumping system.

TESTS

Before the pumping test started, all materials were weighed out and the mixer, pump, and hose system thoroughly wetted. The water, cement, sand, and admixtures were then added in that order, except that the bentonite was first mixed with a small amount of cement and then with the water and sand before introducing the bulk of the cement. After thoroughly mixing, a check consistency test was made using a piano wire torque consistency meter and water added, if necessary to maintain a uniform consistency of 135° ± 15°. During all pumping tests, an attempt was made to operate the pump at a rate of 70 strokes per minute.

After adjustments, the grout mixture was pumped through the hose system and the rate of flow determined. Then the hose system was shut off for a 15-min period and the grout remaining in the mixer allowed to slowly by-pass through the pump and mixer.

TABLE 1.—DATA ON THE PHYSICAL PROPERTIES OF THE FINELY DIVIDED MINERAL ADMIXTURES

Type Admixture (1)	Specific Gravity (2)	Blaine Specific Surface, sq cm per g (3)	Percentage Passing No. 325 Sieve (4)
Fly Ash	2.52	3,580	94
Pumicite	2.34	5,220	98
Loess	2.66	1,800	96
Diatomite	2.25	20,300	97
Bentonite	2.40	155,000	99

After the 15-min period was over, the hose line valves were opened slowly and the grout circulated. If it was possible to recirculate the grout after it had remained in a static condition in the hose line during the 15-min period, the grout was judged pumpable. Three successful pumping tests were made for each combination reported pumpable.

At the end of the pumping tests, consistency, bleeding, and rate-of-flow tests were made and specimens were obtained for compressive strength and time-of-set tests. The line pressures and temperature of the grout were also recorded.

TEST PROGRAM

The program to investigate the pumpability of portland cement-sand grout mixtures was divided into three general phases as follows:

1. Phase I: The effect on the pumpability of grouts containing the three types of sand, each of which contained different percentages of materials passing the No. 100 sieve, was determined.

2. Phase II: The effect of various admixtures on the pumpability of grouts containing the three types of sand with a nominal 0% passing the No. 100 sieve was determined.

3. Phase III: The effect of various admixtures on the pumpability of grouts containing two of the sands with a nominal 10% passing the No. 100 sieve was determined.

Phase I.—The first part of this phase consisted of tests to determine the maximum quantity of natural sand that could be pumped, using six different gradations of sand. These gradations were all basically similar except that the percentages passing the No. 100 sieve were 0%, 5%, 10%, 15%, 20%, and 25%. Table 2(a) gives the results of these tests. It can be noted that the sand content

TABLE 2.—PHASE I - INVESTIGATION OF SAND-CEMENT GROUTS WITH VARYING PERCENTAGES PASSING THE NO. 100 SIEVE

Sand Percentage Passing No. 100 Sieve (1)	Proportions by weight Based on One Part of Cement		Time of Set, in hour		Bleeding, in percentage (6)	Cube Compressive Strength, psi	
	Sand (2)	Water (3)	Initial (4)	Final (5)		7 Day (7)	28 Day (8)
(a) Grouts Containing Natural Sand							
0	2.0	0.63	4	7+	1.2	1750	3500
5	2.50	0.72	4	8+	1.6	1350	2800
10	2.50	0.76	5	8+	1.7	1550	3050
15	2.75	0.82	4	8+	1.7	1400	2700
20	2.75	0.82	4	-	2.7	1300	2500
25	3.00	0.87	4	7+	3.5	1050	2100
(b) Grouts containing Limestone Sand							
0	1.75	0.66	6+	17-	0.9	1795	3780
10	3.25	1.08	7+	18	1.3	660	1405
25	7.00	1.95	7+	70-	1.7	160	340
(c) Grouts containing Traprock Sand							
0	1.75	0.68	5	16-	1.8	2020	4565
10	2.00	0.72	6	21-	1.6	1315	3180
25	2.25	0.83	6	17-	2.1	1265	2830

of the grouts found pumpable varied from two parts for a sand with 0% passing the No. 100 sieve to three parts for the sand with 25% passing the No. 100 sieve.

The second part of Phase I included tests to determine the limits of pumpability of a limestone sand [Table 2(b)]. Based on the work done on natural sands, it was decided to use only the sands with 0%, 10%, and 15% passing the No. 100 sieve. The tests indicated that the parts of manufactured limestone sand that could be pumped increased considerably with an increase in material passing the 100-mesh sieve, varying from 1.75 parts for 0% to 7.0 parts for 25%.

The third part of Phase I was performed to furnish information as to the maximum pumpability of three gradations of a sand with a higher than normal specific gravity. Traprock sand was used in gradations having 0%, 10%, and 25% fine material passing the No. 100 sieve [Table 2(c)]. As in the pumping

test of the other sands, the quantity found pumpable varied from 1.75 parts for the 0% fines to 2.25 parts for the 25%. The relation of the three types of sand to parts found pumpable are shown in Fig. 3.

Conclusions that may be drawn from this phase of the investigation are as follows:

1. It is feasible to pump sanded grouts.
2. Two parts of natural sand and 1.75 parts of limestone or traprock sand, containing a nominal 0% of material passing the No. 100 sieve, based on part of cement were found to be pumpable.
3. An increase in the material passing the No. 100 sieve from 0% to 25% allowed only an increase of one part of natural and 0.5 parts of traprock sands, while the parts of limestone sand increased from 1.75 to 7. An analysis of the material passing the No. 100 sieve indicates that the limestone sand fines have about three times the surface area and seven times the amount of minus No. 325 material as does the traprock or silica sand. This is probably the reason for the difference in the quantity of sand found pumpable.

Phase II.—The three types of sand used in this phase of the investigation were similar to those used in the first part of phase I for each type of the three sands test-pumped.

In the first part of this phase, natural sand with a nominal 0% passing the No. 100 sieve and with the addition of various percentages of four admixtures, fly ash, loess, diatomite, and pumicite, was test-pumped. The results in Fig. 4 show that increasing the amount of admixtures allowed an increase in the amount of sand that could be pumped. As the diatomite had a specific surface about ten times that of the loess, it would appear that this increase is dependent on the fineness of the admixture.

In the second part of this phase, limestone sand with the nominal 0% passing the No. 100 sieve and with the addition of percentages of fly ash and loess was test-pumped. As in the first part of this phase, the parts of sand found pumpable were slightly greater for the mixtures containing percentages of fly ash than for those containing loess.

In the third part of this phase, traprock sand, deficient in material passing the No. 100 sieve, with the addition of percentages of fly ash was test-pumped. As in the other tests in this phase, the additions permitted an increase in the sand-carrying capacity of the grout mixture. The results of the pumping test for the second and third parts of this phase are plotted in Fig. 5. Table 3(a) gives the pumping-test data for the three types of sand test-pumped in this phase.

The tests conducted in Phase II indicate the following:

1. When a sand is deficient in materials passing the No. 100 sieve, the addition of finely divided mineral admixtures will increase the sand-carrying capacity of the grout mixture.
2. The ability of a finely divided mineral admixture to increase the sand-carrying capacity of a grout appears to be related directly to its fineness.

Phase III.—In the first part of this phase, the same natural sand was used that had been used in the other phases of this program with a gradation having 10% passing the No. 100 sieve. Based on the weight of cement, percentages of diatomite and bentonite were added and their influence on the pumpability, or sand-carrying ability, and the characteristics of the grout mixtures in both plastic and hardened conditions were studied. The results indicated that the

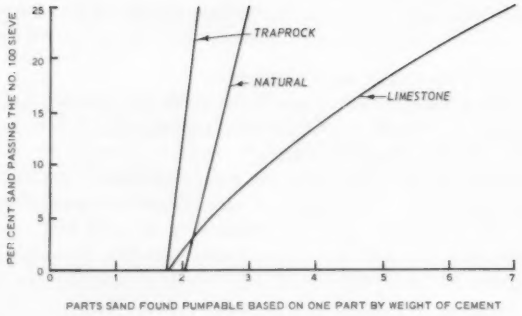

FIG. 3.—PHASE I. THE EFFECT OF FINE SAND ON THE PUMPABILITY OF GROUTS CONTAINING THREE TYPES OF SAND

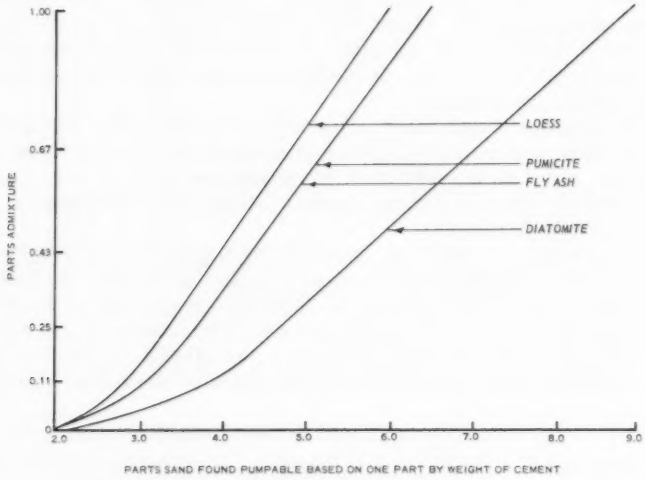

FIG. 4.—PHASE II, PART 1. THE EFFECT OF ADMIXTURES ON THE PUMPABILITY OF GROUTS CONTAINING NATURAL SAND DEFICIENT IN 100 MESH SIZE

TABLE 3.—EFFECT OF ADMIXTURES ON PUMPABILITY

Type Admixture	Proportions by Wt Based on One Part of Cement			Time of Set, in hours		Bleeding, in percentage	Cube Compressive Strength, psi	
	Admixture	Sand	Water	Initial	Final		7 Day	28 Day
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Phase II: (a) Grouts containing Sands with 0% passing No. 100 Sieve								
Natural Sand								
No Admixture	-	2.00	0.63	4	7	1.2	1730	3505
Fly Ash	0.11	3.10	0.88	6	21-	1.5	1195	2210
Fly Ash	1.00	6.50	1.62	7+	26	1.3	435	715
Loess	0.11	2.80	0.88	5+	18-	1.3	1300	2355
Loess	1.00	6.00	1.88	2+	50-	1.2	255	410
Diatomite	0.11	3.90	1.19	6	20-	1.2	585	1035
Diatomite	1.00	9.00	3.18	6+	19-	0.4	55	235
Pumicite	0.11	3.10	0.94	3+	17-	1.8	1095	1965
Pumicite	1.00	6.50	2.04	3+	26	1.9	170	380
Limestone Sand								
No Admixture	-	1.75	0.66	6+	17-	0.9	1795	3780
Fly Ash	0.11	1.90	0.73	6+	18-	0.9	1650	3370
Fly Ash	1.00	5.00	1.58	7+	24	0.9	595	1385
Loess	0.11	1.90	0.74	5	19	0.8	1595	3000
Loess	1.00	4.50	1.62	6+	21	1.0	265	535
Traprock Sand								
No Admixture	-	1.75	0.68	5	16-	1.8	2020	4565
Fly Ash	0.11	1.94	0.73	4+	16-	1.5	1435	3420
Fly Ash	1.00	4.50	1.49	7+	22-	1.4	710	1450
Phase III: (b) Grouts containing Sands with 10% passing No. 100 Sieve								
Natural Sand								
No Admixture	-	2.50	0.76	5	8+	1.7	1540	3030
Diatomite	0.03	3.00	0.95	4+	21	-	860	1840
Diatomite	0.06	3.25	1.01	3+	19	-	745	1900
Bentonite	0.025	4.0	1.35	8+	22-	-	375	670
Bentonite	0.40	32.0	12.17	70+	500+	-	0	0
Limestone Sand								
NoAdmixture	-	3.25	1.08	7+	18	1.3	660	1405
Diatomite	0.11	4.50	1.54	4+	19-	1.7	355	805
Diatomite	1.00	12.00	4.17	3+	82-	0.6	75	255
Fly Ash	0.25	3.75	1.18	4+	20-	1.1	800	1515
Fly Ash	1.50	7.5	2.10	3+	23	0.8	440	1025

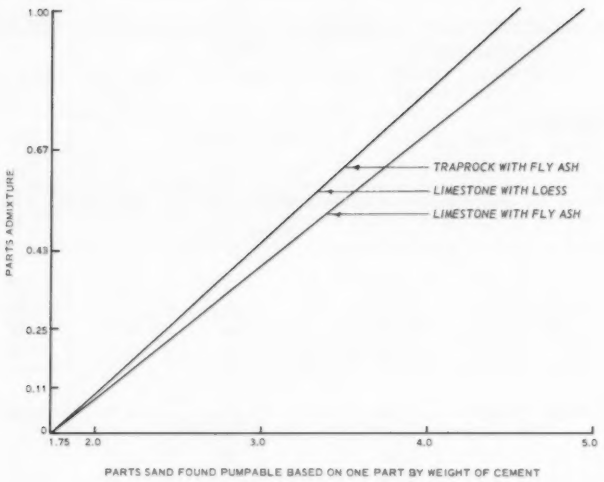

FIG. 5.—PHASE II, PARTS 2 AND 3. THE EFFECT OF ADMIXTURES ON THE PUMPABILITY OF GROUTS CONTAINING LIMESTONE AND TRAP-ROCK SANDS DEFICIENT IN 100 MESH SIZE

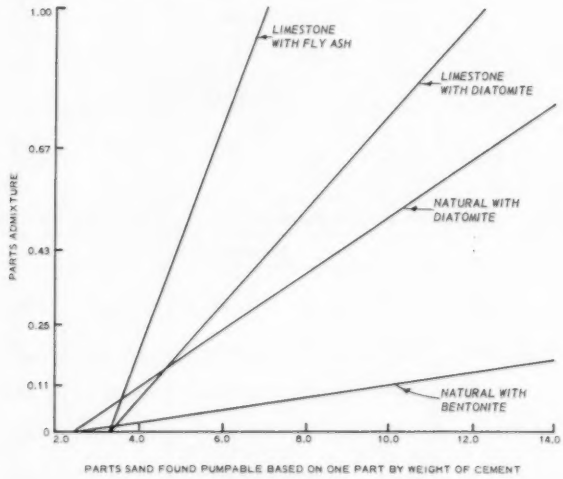

FIG. 6.—PHASE III. THE EFFECT OF ADMIXTURES ON THE PUMPABILITY OF GROUTS CONTAINING NATURAL AND LIMESTONE SANDS WITH A NOMINAL 10 PER CENT PASSING THE NO. 100 SIEVE

addition of percentages of 0.06 parts of diatomite permitted an increase of 0.75 in the parts of sand found pumpable, while an addition of 0.40 bentonite permitted an increase of 28 parts of sand.

The limestone sand used in the second part of this phase was the same as that used in phase I with 10% passing the 100-mesh sieve. Percentages of diatomite and fly ash were added to the grout mixtures, as in the first part of this phase. The addition of one part of diatomite permitted an increase of 8.75 parts of sand found pumpable, while 1.5 parts of fly ash allowed an increase of 4.25 parts of sand. Table 3(b) lists the results of these pumping tests, and Fig. 6 illustrates the results graphically.

This phase of the investigation indicated that the addition of percentages of the various admixtures will increase the sand-carrying capacity of a sand-cement grout when sands with sufficient fines are being used (10% passing the No. 100 sieve).

It is believed that while the large addition of bentonite in these tests did permit a large increase in the amount of sand, the low strength that results when large quantities of sand are used would allow its use only in special cases. (This is not to imply in any sense that small quantities of bentonite are not useful in pressure grouting.) In addition to the admixtures mentioned previously, IP Fluidifier and Methocel were investigated. Both materials permitted an increase in parts of sand found pumpable.

CONCLUSIONS

A number of conclusions can be drawn from the data obtained in this investigation:

1. Sand deficient in material passing the No. 100 sieve can be successfully pumped in grout mixtures without the use of an admixture.
2. Concrete sands can be used in portland-cement grout mixtures provided they are scalped over the No. 16 sieve.
3. An increase in fine material in the minus No. 100 sieve size will permit an increase in the quantity of sand found pumpable.
4. Manufactured sands can be successfully pumped in portland cement-sand grout mixtures.
5. The specific gravity of the sands included in this test program had little or no effect on the pumping characteristics of the grout.
6. In using sand deficient in minus No. 100 sieve size material, the addition of a finely divided mineral admixture increases the sand-carrying capacity of the grout.
7. The addition of a finely divided mineral admixture to a normal portland cement-sand grout will increase its sand-carrying ability.

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SYMPOSIUM ON GROUTING: CONSTRUCTION OF ROCKY REACH GROUTED CUTOFF

By W. F. Swiger,[1] F. ASCE

FOREWORD

This paper is one of several papers that constitute the Symposium on Grouting which occupied two sessions of the June 1960 meeting of ASCE in Reno, Nevada. Credit for organizing this symposium program goes to a Special Grouting Symposium Committee consisting of Stanley J. Johnson, F. ASCE, Chairman, T. William Lambe, F. ASCE, and W. K. Seaman, F. ASCE.

It is believed that these several papers constitute a significant contribution to engineering practice in the art and science of stabilizing foundation materials by grouting. This is a field in which there appears to be increased interest among foundation engineers and one which is expected to become more important as time goes on and the need to build structures on difficult soil conditions increases.

The papers of the symposium are concerned with cement, clay, and chemical grouting of seams, cavities, and granular materials. The papers illustrate the increased use of chemicals for grouting purposes and also their use with more conventional grouting materials such as clay and cement; grouting of overburdened materials which is rapidly becoming a more definite art and science and which, to some degree at least, is susceptible to laboratory studies; the grouting of a cut-off in granular materials to prevent underseepage beneath a major hydroelectric project (perhaps the first application of this type of grouting in the United States); and the grouting of subsurface voids and solution channels in limestone areas subject to large subsidence.

Note.—Discussion open until September, 1961. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

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Organized in 1952, the Grouting Committee with Task Committees on Cement, Clay, and Chemical Grouting has sponsored groups of papers or symposia occupying the sessions of meetings of the Society in 1954 and 1956, as well as 1960. Papers on grouting which have been published in the Journal of the Soil Mechanics and Foundations Division are as follows: Nos. 1145, 1204, and 1544 to 1552 inclusive. The report of the Task Committee on Chemical Grouting appears as Paper 1426 in the November 1957 Journal. A draft of a report on clay grouting has been prepared by the Task Committee on Clay Grouting, and the report on cement grouting prepared by the Task Committee on Cement Grouting is substantially in its final form.

The Administrative Committee on Grouting takes this opportunity of expressing its appreciation to the task committees for the excellent manner in which they have carried out their assignments and to the individuals who, as writers of papers, have made significant contributions covering the practices of grouting in various fields.

Raymond E. Davis, F. ASCE, Chmn., Admin. Committee on Grouting, Prof. of Civ. Engrg., Emeritus, Univ. of California, Berkeley, Calif.

SYNOPSIS

Grouting is a commonly used and long accepted practice for cutting off or controlling seepage through rock foundations or abutments of dams. Its use for effecting a cutoff in alluvials, however, is almost without precedent in the United States, although it has been used on several projects in Europe and in Canada. At the Rocky Reach project on the main stem of the Columbia River north of Wenatchee, Wash., the left abutment is a broad, high terrace overlying a buried channel. The basal member of the sediments forming this terrace is a series of openwork gravels open to the river. This paper describes the design and construction, by grouting, of the cutoff constructed across this gravel to control seepage through it.

The major portion of the cutoff consists of three rows of holes, the rows being spaced 10 ft on center, and the holes along the rows spaced 12 ft on center. Near the abutment of the dam, five rows were used and a single row was used at the easterly end of the cutoff across a subsidiary channel. Grouting was by stages of 5 ft height starting at the top of the stratum.

The primary grout used was cement mixed with clay or bentonite to reduce sedimentation. On completion of cement grouting, intermediate holes were drilled, generally along the center line of the cutoff, and grouted using AM-9, a chemical grout, in order to fill and seal sand strata which were so fine that they refused to accept cement grout. The chemical grout AM-9 is a recently developed material of water-thin viscosity when injected, which gels at a predetermined and controllable time by polymerization.

Observations of piezometric levels prior and after the work indicate that an effective cutoff was constructed.

The Rocky Reach Hydroelectric Power Project is located on the main stem of the Columbia River about 9 miles above the city of Wenatchee in north central Washington. As shown on Fig. 1, the power house is located on the west,

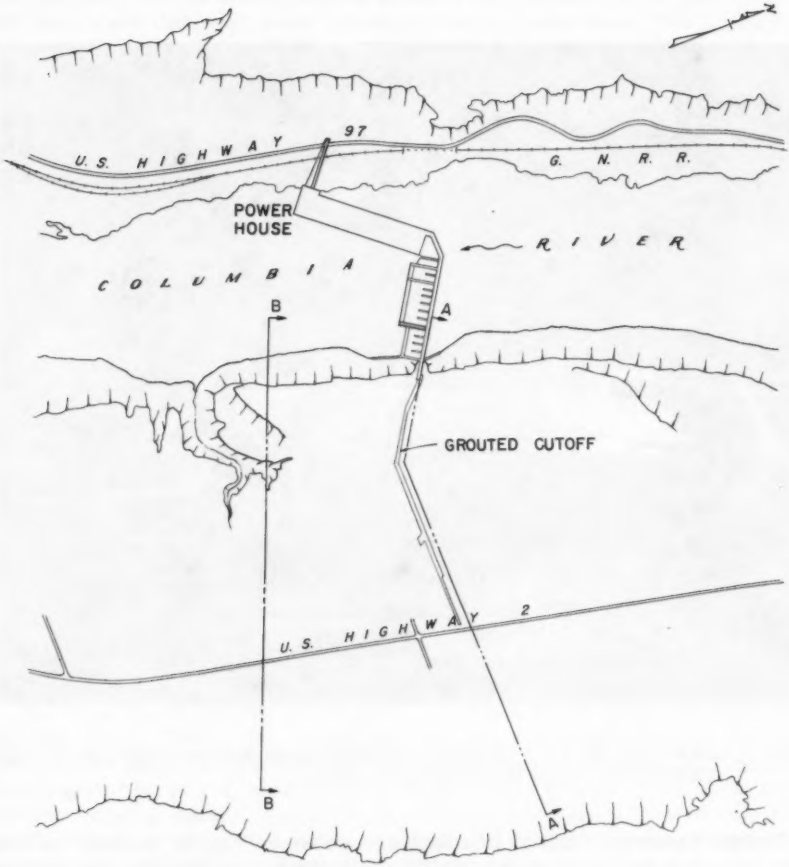

FIG. 1

or right bank. Initially, seven units with a total generating capacity of 775,000 kw at 93 ft gross head will be installed with provision for four additional units which, when installed, will give a total capacity of 1,215,000 kw. The spillway of twelve radial gates, each 50 ft wide by 58 ft high, adjoins the power house at its northeast end and extends in a straight line across the river (Fig. 2).

Spillway capacity, including flow through the power house, is 1,200 cfs. The flow of record, at this location, is 718,000 cfs.

Here, the Columbia occupies a tremendous canyon, nearly 1 mile wide at its bottom and more than 3,000 ft deep. At the dam site the river itself is a relatively shallow, fast flowing stream about 1,300 ft wide. It lies on a high bedrock shelf, close against the western wall of the canyon, and is bordered on the east by a high, broad terrace which extends more than 3,500 ft from the river edge to the canyon wall on the east at an elevation of about 140 ft above low water in the river.

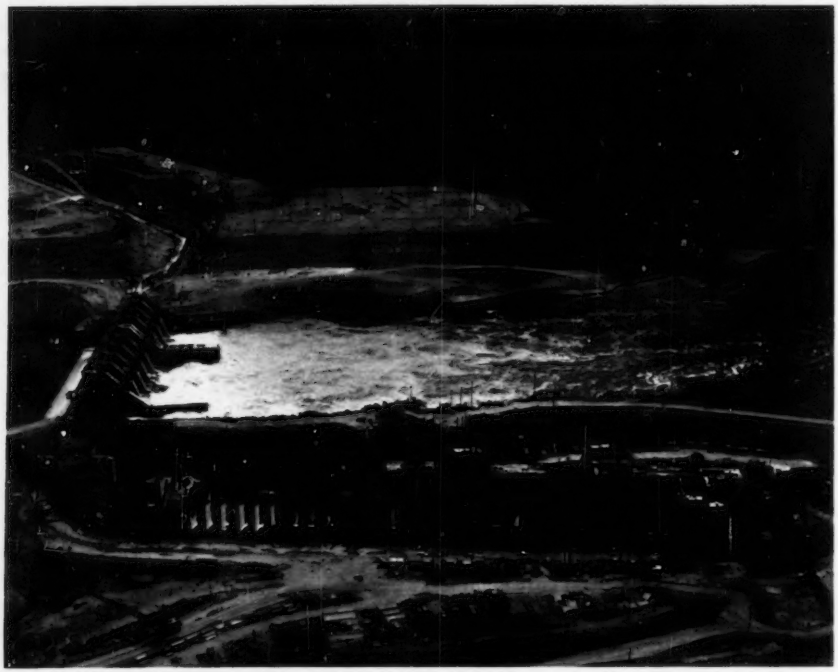

FIG. 2.—GENERAL VIEW OF PROJECT LOOKING EAST SHOWING SPILLWAY AND TRENCH IN WHICH GROUTING WAS DONE

Typical cross sections of this bank are shown on Fig. 3. Section AA was taken on a line extending approximately along the face of the dam, and section BB at a point about 1,200 ft downstream from the dam. The bedrock under the bank surface is extremely irregular and vertical cliffs of more than 60 ft high were found during construction.

The lower gravels are the oldest of the three formations constituting this terrace. Apparently at some time during the Pleistocene they formed a broad terrace lying west of the antecedent river which was then cutting on bedrock along the eastern edge of the canyon at an elevation more than 130 ft below the

present level of bedrock in the river channel. The edge of this terrace runs southwest-northeast at an angle of about 25° with the course of the present river and terminates against a bedrock ridge downstream of the line of the cutoff. This condition ended with blocking of the Columbia downstream of the site, creating a long glacial lake which filled with a series of varved clays. Remnants of these clays have been traced to nearly 300 ft above the present river elevation. The varved clays are tough and highly precompressed. They contain innumerable thin varves of silt and fine sand and occasional sand members a few inches to as much as 10 ft in thickness. With the resumption of flow through the canyon of the Columbia, several hundred feet of these clays were removed by erosion and the upper, most recent gravels laid over them. The upper gravels are about 80 ft thick. They consist of a series of river-laid, heavy gravels, cobbles and boulders up to 10 cu yd in size, with some patches of openwork materials.

The primary problem in developing the site was controlling seepage around the end of the dam through the lower gravels. Upstream of the dam, the bedrock surface along the edge of the bank drops, while the surface of the lower gravels rises. Therefore, the thickness of the gravels exposed to the reservoir increases significantly upstream. Downstream exposure of the gravels to the river bed is restricted since the top of the gravel lowers whereas bedrock in the river, except for a few low spots, generally remains high. As can be seen from the sections, the bottom of these gravels extends to more than 150 ft below tailwater level in the river.

Examination of the lower gravels in cuts down the face of the river bank and during excavation for the first stage of construction showed them to be primarily a series of openwork gravels, with occasional sandy strata. They showed extensive cross-bedding, with many of the earlier beds truncated by later deposits. Estimates of the coefficient of permeability of these materials, based on well tests, were inconclusive because of the small drawdowns. It was concluded from visual comparison that permeability would be about the same as for apparently similar gravels in the abutments of the McNary Dam[2] which have an average coefficient of permeability of 0.2 fpm, based on observed seepage after completion of the dam. A design value of 2 fpm was assumed. Based on this coefficient, it was planned to use an upstream blanket and a drainage system of a series of relief wells discharging through a tunnel in the east bank. In these preliminary studies, consideration was given to the use of a grout curtain as an alternate and an extensive grouting test was undertaken.

Construction of the project started with the erection of a cofferdam along the east bank. Observation of seepage into the cofferdam through the lower gravels indicated coefficients of permeability between 10 fpm and 15 fpm, which is extremely large and significantly exceeded the 2 fpm assumed in the initial design. After a careful review of the seepage problem in view of this large coefficient of permeability, a grouted cutoff was decided. It was concluded that this cutoff must satisfactorily accomplish the following:

1. Control seepage through the lower gravels in such a manner that a process of underground erosion, or piping, could not develop.
2. Limit seepage sufficiently so that pressures under the downstream bank would not cause slides.
3. Limit seepage to economically acceptable values.

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2 "McNary Dam-Design from Technical Considerations," by H. L. Drake, G. C. Richardson, H. M. Rigler, and R. A. Schuknecht, Proceeding, ASCE, Vol. 81, Proceedings-Separate No. 787, September, 1955.

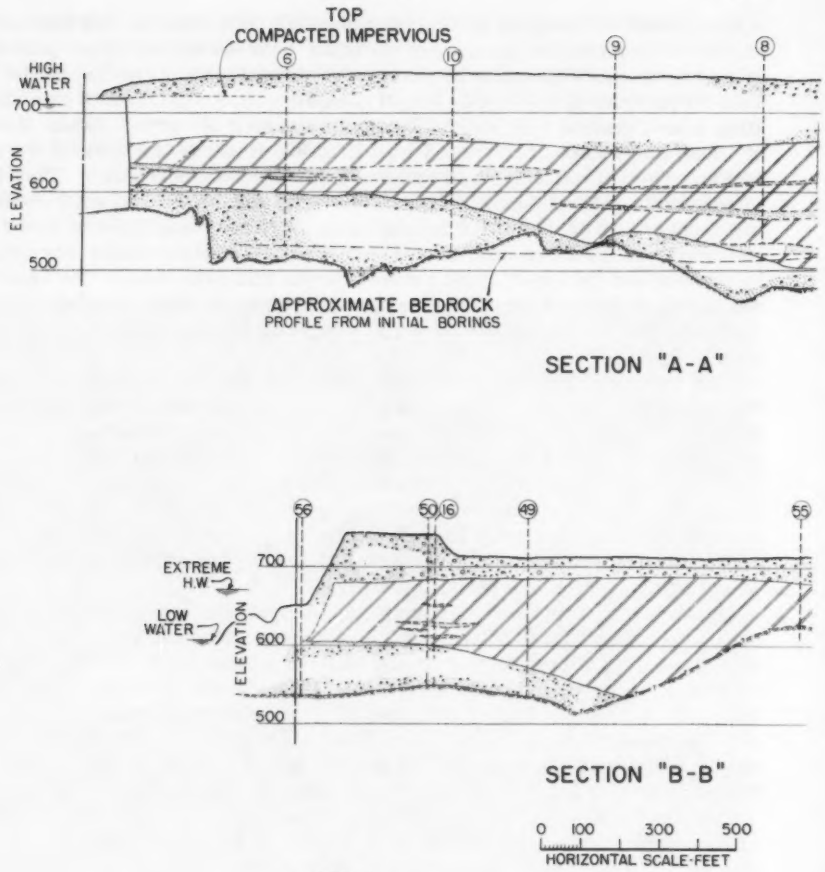

FIG. 3

As shown on the sections, the eastern series of gravels are thin, river laid deposits. West of these are the thick series of openwork materials which are open to the river. Based on the seismic surveys and original borings, it was thought the western series terminated about 1,500 ft from the end of the dam. Seepage studies indicated that, if this were blocked by a suitable cutoff, seepage around the end of the cutoff would be acceptably small. Actually as the work progressed it became apparent from piezometric levels that the high bedrock ridge which formed the eastern edge of the old terrace was continuous and in contact with the varved clays. Consequently it was possible to terminate the main body of the cutoff where it crossed this ridge at a point about 1,200 ft from the dam.

The gravels in the secondary channel just east of this bedrock ridge were found to be interconnected with the gravel of the eastern channel, but are apparently of limited extent. They were probably deposited as a bank of openwork materials in the lee of the bedrock high shown on section AA at boring

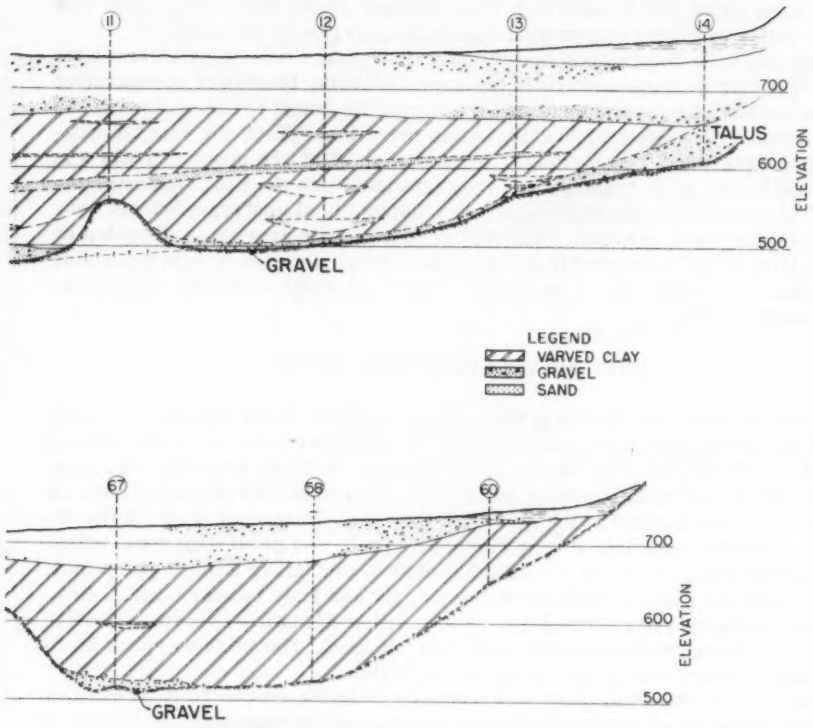

FIG. 3.—*CONTINUED*

CD-11. A secondary cutoff consisting of a single line of cement grout holes was constructed across this channel terminating where the gravels pinched out at about 1,800 ft from the dam.

The cutoff was not conceived as a totally impervious structure. Accordingly, it was necessary to provide for discharging such seepage as would pass through or around it. A 60 in. diameter, open joint and concrete drain, with suitable filters, was constructed, extending about 400 ft downstream of the face of the dam. This was located against, the drains through, the downstream training wall. For another 600 ft, the river bank was excavated to bedrock through water and a graded filter drain placed along the face of these lower gravels. This filter grades by stages from coarse gravel to heavy riprap at the river face.

It was convenient and economical to drain the varved clays by carrying these filters upward across the face. A sand seam about 8 ft in height, in the varved clays, was exposed during the excavation for the first stage. This sand mem-

ber extended about 400 ft upstream from the face of the dam. This sand was covered with compacted impervious material upstream of the dam.

Blanketing with impervious materials and drainage were considered adequate to provide seepage control in the upper gravels. Studies of comparative cost showed that, by excavating along the line of the cutoff to the surface of the clay, savings in the costs of drilling would more than offset the cost of excavation and backfilling. Accordingly, a partial cutoff of the upper gravels was used, constructed as an underground dam extending into the bank approximately 2,000 ft, rather than the somewhat more conventional upstream blanket over the face of the upper gravels. Drainage of these upper gravels is provided by an open joint drain with suitable filters constructed along the downstream side of this cutoff for a portion of its length. A typical section through these cutoffs is shown on Fig. 4.

PRELIMINARY GROUTING TESTS

For the preliminary grouting test, a ring of grout holes spaced 5 ft apart around the circumference of a circle 30 ft in diameter were to be drilled and grouted in 5 ft stages from the top of the gravels down to bedrock. Permeabilities before and after grouting were to be determined by pump testing a 12 in. diameter well located at the center of the ring. To permit study of the effect of a variable spacing, it was planned to drill and grout alternate holes, make a pump test, grout the intermediate holes, and make a third pump test.

The grout used was a mixture of cement, fly ash and "intrusion aid." Proportions were generally about 3:1:2 water-cement-fly ash by volume. Grouting was to refusal, with pressures up to 300 psi used. The work was done from the ground surface and caused no apparent heave.

Actually, only the first stage of grouting with holes spaced 10 ft on centers was done. The pump test, before grouting, showed a drawdown measured in the well of about 4 ft when pumping at a rate of 600 gpm. Following the first stage of grouting, the measured drawdown was 70 ft while pumping at a rate of 140 gpm. The work was expensive and significant improvement in observable results was doubtful; accordingly, the second stage was not undertaken. Grout travel of as much as 25 ft was noted in the work.

On the basis of later knowledge, it is believed the procedure of pump testing from the same well before and after grouting is a doubtful practice. Development of the well tends to create an openwork zone around it. Despite preventive measures, such an openwork zone is apt to be filled by grout resulting in misleading results in the later test. This test did, however, show that grout could be injected in adequate quantities, that significant reductions in permeability would result and that radial travel of grout of as much as 25 ft would occur. The test also established a practicable method of drilling and grouting.

Construction of the eastern end of the concrete dam abutment required use of a sheet pile cofferdam driven through the lower gravels to permit excavating to about 20 ft below ground water level. Considerable difficulty was experienced with this sheeting hanging up on boulders before reaching bedrock. The contractor elected to grout along this line of sheeting and excavate within the protection of the grouted gravel. Two rows of grout holes were used, spaced respectively 4 ft and 8 ft from the line of sheeting, the outer row being grouted first. The grout was cement with about 10% by weight of bentonite. Churn drill holes were used. Excavation extended about 9 ft below the tip of the sheeting

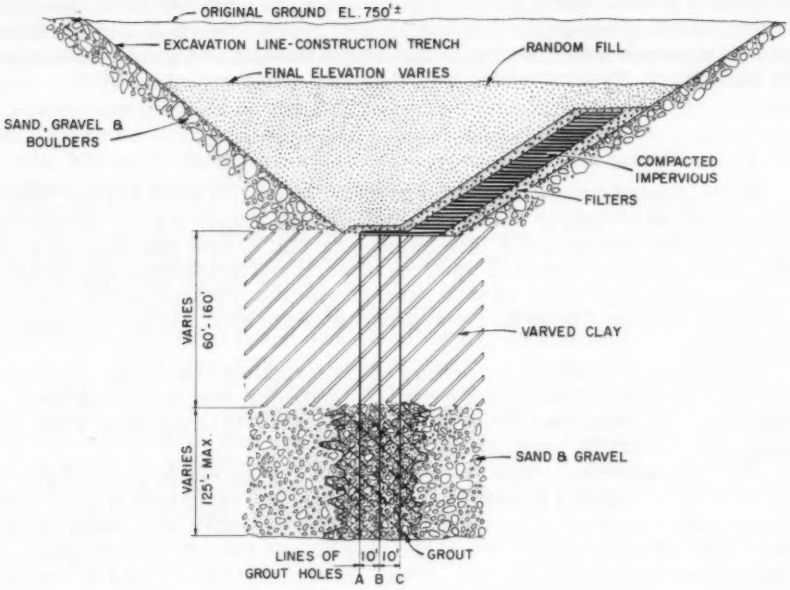

FIG. 4

FIG. 5.—GRAVELS GROUTED TO FACILITATE CONSTRUCTION OF DAM ABUTMENT

in grouted gravel, with a ground water level about 20 ft above the lowest point of excavation. Seepage through the grouted gravel was small and no difficulty was experienced with stability. Fig. 5 shows this grouted gravel when exposed by excavation. Hydraulic head behind these gravels was about 20 ft.

DESIGN OF GROUTED CUTOFF

While grouted cutoffs have been used for nearly 20 yr in Europe, they are still in the development stage, especially in the United States. Accordingly, in undertaking the design of this cutoff, a careful review was made of a number of references relating to this type of work, and procedures and techniques used on previous projects were studied. From these references, the more pertinent of which are listed in the Appendix, certain basic design concepts were developed.

Although the average permeability of the formation was large, the arrangement and physical properties of generally lenticular, individual beds were such that wide and erratic variations in permeability could be expected within short distances, especially vertically.

Previous experience with grouting of granular materials and data reported by other investigators showed that even moderate variations in permeability would cause the grout to move principally into the more permeable members with little intrusion of grout into the less permeable. To assure good grout distribution throughout the mass, it was anticipated that it would be necessary to limit the vertical height to be grouted in one stage to minimize the range of permeabilities to which the grout would be exposed, that several attempts should be made to grout each zone and that the procedures, arrangement of holes and sequence of grouting should be such that the more open, permeable zones would be filled first and the less permeable zones filled by later grouting, either from the same hole or from adjacent holes. Although available data showed a marked decrease in permeability could be obtained with close spacing on a single line of holes, it was considered doubtful whether results approaching a complete cutoff could be achieved.

Even with these precautions and procedures, it was extremely doubtful that all voids, even of the coarser material, would be filled. Grout consisting of finite particles such as cement, or clay and cement, can be injected only into material having pore spaces several times the size of the coarser materials in the grout. Such grouts would not penetrate sands containing more than about 10% of material finer than 1 mm. Accordingly, it was to be expected that sand members in the alluvials, in general, would not be grouted if cement or cement-clay grouts were used alone. By using a cutoff of substantial width, the paths of percolation which remained could be made tortuous and long so that the velocity of flow through them would not erode the grout. Also sand members, of limited areal extent, might be completely enclosed within grouted material.

By grouting the downstream row first and taking care to inject large quantities of grout into these holes, the coarsest, most open materials into which conceivably the finer sands might move by internal piping, could be filled with grout. This would provide a wide zone of partially grouted material which would act as a graded filter to prevent piping. European practice has been to provide a cutoff having a width of 25% to 30% of the head of the structure and such a width seemed adequate to assure stability over the life of the project.

A minimum of three rows of grout holes was considered necessary to achieve these results and desired width. The outer rows should be grouted first and the inner row last to assure reasonably successive filling of the less pervious zones.

Neat cement grout is not satisfactory for grouting alluvials because of sedimentation or settling within the soil mass, especially within the coarser materials. Mixing of the cement with other materials is necessary to form a stable grout to fill the voids completely after it has set.

Experimental grout mixes were made using the local clay available from the varved clay deposits on the site, fly ash, Kenite K-75, a processed diatomaceous earth which is produced near Quincy, Wash., and a processed pumicite from near the Priest Rapids dam site. Each material produced grouts of acceptable physical properties as measured by stability, strength, density, toughness and erosional resistance. Preliminary estimates indicated the local clay cement grout should be the least expensive. All of these materials contained about 2% to 5% of material coarser than a 200 mesh screen, and it was believed this would be satisfactory. It was concluded that the use of extremely finely ground cement was not warranted. However, by selecting the cement mill, it would be possible to obtain, without paying a premium, a Type II cement having a Blaine specific surface of at least 3,200 sq cm per g, which is a finer than average material.

To assure intimate and thorough mixing of the cement and clay, use of high speed, rotary pump type mixers was considered necessary.

Based on these general design criteria, specifications were prepared for obtaining bids. The main body of the grout curtain was to be three rows of holes, except that for the first 200 ft out from the dam abutment, five rows would be used. These rows originally were spaced 8 ft on centers. This was later changed to 10 ft on centers from hole 32 eastward.

Three different and equally acceptable methods of doing the work were specified. Method No. 1 contemplated drilling for and setting a casing to bedrock. Grout would thus be injected in zones of 3 ft to 5 ft height, beginning at the bottom. The casing would be perforated in this stage using a knife. The entire thickness of gravel would then be regrouted in narrow zones, again starting at the bottom. For this stage and any later stages of grouting, the casing would be perforated using a gun perforator. This would assure that any grout collar immediately adjacent to the casing was shattered and thus permit entrance of the grout to less pervious zones which possibly were not grouted in the first stage grouting. Three stages of grouting were specified at the bedrock and at the clay gravel interface, since these are the most difficult places to obtain good results. Double packers would be set in the casing just above and below each zone to be grouted.

Method No. 2 was based on the procedures developed in the preliminary test program. The casing would be set through the clay to the top of the gravel. The gravel would be drilled using rotary equipment through a zone of 5 ft, starting at the top of the gravel. This drilled hole would be thoroughly washed to remove any fine material and cuttings and the zone grouted. After the grout had hardened, the hole would be redrilled through the grouted material and 5 ft deeper into the gravel and the procedure repeated. It was anticipated that zones which might have refused grout on the first pass would be picked up in later redrilling and grouting steps. It was specified that, after bedrock was

reached, the hole should be drilled completely open from top to bottom of the gravel and grouted at least three times.

Method No. 3 was based on the procedures developed and used in Europe by The Rodio Group. A small diameter grout pipe is set to bedrock in the drilled hole. This grout pipe has holes drilled through it at intervals of about 12 in. These holes are covered on the outside by a short rubber sleeve which acts as a valve. The annulus between the pipe and the drill hole is filled with a weak, brittle grout. Grouting is by stages from the bottom up, using a double packer. Individual stages are usually about 1 ft high.

The basic grout was established as a clay-cement mixture of one part cement to one and one-half parts clay, using the local clay. Alternative grouts of fly ash-cement mixtures, or K-75 cement mixtures, were permitted and bids were taken on these materials.

Customary European practice is to pump a predetermined quantity of grout into each zone, letting pressures build up to whatever amount is necessary and accepting whatever ground heave occurs. Reportedly pressures as high as 1,200 psi to 1,500 psi have been used. This procedure was considered undesirable, especially for this site. Heave results where thin lenticular bodies of pure grout are created, which is a waste of grout. Also, heave, if significant, might cause cracking of the clays overlying the gravel formation. Accordingly, it was specified that grouting should be done at rates which would not require excessive pressures until refusal was reached or until a maximum quantity of grout had been injected. These originally were set at 200 psi and a maximum quantity of 60 cu ft of grout per linear foot of hole in the zone being grouted. Refusal was defined as 1 cu ft take in a period of 5 min at maximum specified pressure.

Some difficulty with washing and displacement of the fresh grout was anticipated as closure of the grout curtain was approached when a substantial head differential built up across the curtain. Use of silicate admixtures to give a controlled gel time appreciably shorter than that of the standard cement grout was considered a possible requirement. Provision was made in the specification for use of silicates mixed with the grout and an agreement was negotiated with The Rodio Group to permit use of sodium aluminate as a gelling agent, since this compound which permits the use of a single injection process is patented by this group. Use of silicate grouts would also have permitted grouting finer materials than could be penetrated by the cement-clay grout.

It was specified that grouting should start at the abutment and proceed to the east in order that final closure would be made at the farthest possible distance from the river. Further, a dirty gravel having low permeability was used for fill behind the upstream training wall and as a blanket for approximately 700 ft upstream from the dam. This blanket of low permeability material connected with a sheet pile cutoff driven by the contractor in constructing the cofferdam for the first stage and which extended along the river edge for an additional 750 ft. This was done to develop a long seepage path around the abutment during construction of the grouted cutoff to minimize difficulties with washing of the grout.

Specifications required that the first step should be construction of two test patterns approximately 50 ft long at hole spacings of 12 ft and 15 ft as a basis for establishing the basic hole spacing for the remainder of the cutoff.

Grouting is an extremely variable process in which the quantity of grout injected, hole spacing and other factors may vary sharply from point to point in

the cutoff. To assure a satisfactory job, it is necessary that the work be observed continuously and adjustments made in procedures, mixes and other factors as the work progresses. Bids were taken for the work on the basis of a lump sum for setting up and dismantling the plant, unit prices for each item of material supplied and unit prices for each item of work done, such as per foot of drilling or per cubic foot of grout mixed and placed. In this manner, while the contractor was made wholly responsible for supplying the plant and executing the work, the engineers retained complete control of the many factors which would affect the quality of the completed structure. These included location, spacing, sequence of drilling various holes, and control of grout mixes, of quantity of grout injected, and of pressures used.

The contract was let to a contractor who planned to use Procedure II.

CONSTRUCTION OF THE CUTOFF

Work was started on March 4, 1958, with construction of the grout plant and erection of temporary wooden working platforms along the line of the cutoff. Injection of grout into the holes of the two test areas was started May 14, 1958.

Drilling.—For each grout hole, a casing was set through the clay and grouted in. The gravels were then drilled in stages of 5 ft from the top down and cleaned by thorough surging and washing. A packer was then set near the top of the casing and the hole grouted. After 2 hr to 4 hr, the hole was washed to remove partially hardened grout. Washing was carried not deeper than 6 ft above the top of the stage which had been grouted. After the grout had hardened a minimum of 36 hr, the hole was reentered and the next lower stage drilled and grouted. Bedrock elevation was checked as the work progressed by drilling 5 ft to 20 ft into it in selected holes.

The contractor planned to use a continuous flight auger to drill through the varved clay for setting the casing. This method was unsuccessful, possibly due to the moisture content of the material. After some experiments, a Failing "1,500" truck-mounted drill equipped with "N" drill rods and a 5 in. diameter "fishtail bit" was adopted. Fig. 6 shows the grouting in progress. Note the plank floor. The pipes descending the right bank carry grout, water, and compressed air. One of the two Moyno pumps used to inject grout is shown to the right of the nearest drilling rig.

Water was the only fluid used for drilling and was used at a rate which maintained a slurry of the material in the hole to prevent the inflow of sand from sand members in the clay. On completion of the hole to the lower contact of the clay, the drill tools were withdrawn, the casing was lowered into the hole, and the annular space between the exterior of the casing and the drilled hole was filled to the ground surface with a heavy cement base grout. This was forced through the bottom and perforations and upward around the exterior of the pipe. All casing was left permanently in place. The casing used was 4 in. ID, 14 gage welded steel pipe with a bell on one end. The 20 ft joints of casing were joined by arc welding. The lower 2 ft of the bottom length of casing were perforated.

The specifications were based on using tricone rotary bits with water for drilling the gravels. The contractor requested, as an alternate, approval of percussion type drills using 3-1/8 in. bits and drilling with air, but with provision for washing with water as necessary. This was granted after procedures were worked out and adopted which assured thorough washing and clean-

ing of the hole. Both types of equipment were used more or less interchangeably on the project. However, even the rotary drilled holes were surged with air to clean them since observations indicated better cleaning of the walls of the holes when both air and water were used. While drilling, the hole was washed with water or air as necessary to clear cuttings. The hole was permitted to fill with water by seepage and then surged with air with the bit near the bottom of the hole. These procedures were repeated until the hole was clean and ready for grouting. Surprisingly, little difficulty was experienced with hole caving.

FIG. 6.—GROUTING IN PROGRESS

In the course of the work, the contractor experimented with several other means of drilling in hopes of developing cheaper, more effective methods. These included diamond and hard-faced steel bits. None of these were as effective as the rotary and pneumatic equipment.

Cement Grout Plant.—The equipment for mixing the cement-based grout was housed in a stationary plant located on the edge of the trench at a point equidistant from the ends of the cutoff. The plant and much of the equipment were designed by the contractor and permitted the entire batching and mixing operations to be done by one operator from a central control panel. A flow diagram of the plant is shown on Fig. 7.

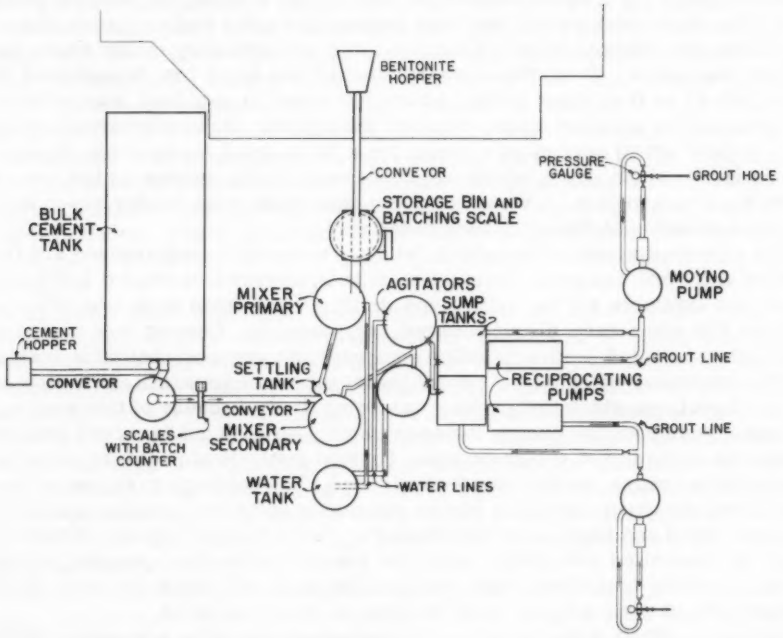
BENTONITE HOPPER
CONVEYOR
PRESSURE GAUGE
GROUT HOLE
STORAGE BIN AND BATCHING SCALE
BULK CEMENT TANK
MOYNO PUMP
MIXER PRIMARY
AGITATORS
SUMP TANKS
CEMENT HOPPER
GROUT LINE
SETTLING TANK
RECIPROCATING PUMPS
CONVEYOR
CONVEYOR
GROUT LINE
SCALES WITH BATCH COUNTER
MIXER SECONDARY
WATER TANK
WATER LINES

FIG. 7

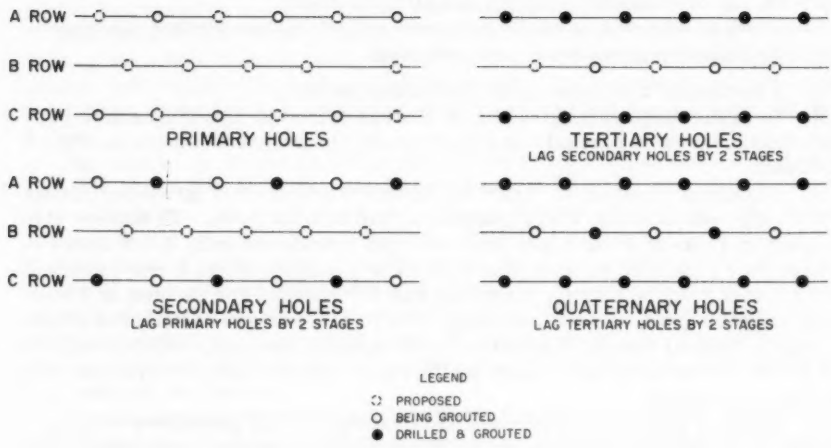
A ROW
B ROW
C ROW
PRIMARY HOLES
TERTIARY HOLES
LAG SECONDARY HOLES BY 2 STAGES
A ROW
B ROW
C ROW
SECONDARY HOLES
LAG PRIMARY HOLES BY 2 STAGES
QUATERNARY HOLES
LAG TERTIARY HOLES BY 2 STAGES
LEGEND
PROPOSED
BEING GROUTED
DRILLED & GROUTED

FIG. 8

The clay or bentonite was mixed in the primary mixer with the volume of water required for a water-cement ratio of 2:1 by volume, the thickest grout used. The clay-water slurry was then pumped through a sedimentation chamber to remove unmixed lumps of clay, and into the secondary mixer where the cement was added. From the secondary mixer the grout was transferred to one of two 60 cu ft agitator tanks. Additional water, if required, was added to the grout in the agitator tanks. Each of the agitator tanks discharged into a 20 cu ft tank which served as a sump for a 10 in.-by-3 in.-by-10 in. duplex, double action slush pump, which supplied grout to the header in the trench through a 2 in. pipeline. A return line was connected to the header which permitted circulation of the grout back to the sump.

The conveyor systems throughout the plant were electrically driven and the mixers, agitators, and pumps were pneumatically powered. A total of 2,400 cfm of air was available for the mixing and drilling operations from one 600 cfm and two 900 cfm rotary diesel powered compressors. Cement was stored in two trailer-mounted, horizontal silos equipped with automatic batching scales.

The grout plant being on the edge of the trench was approximately 80 ft above the platform from which the grouting was being done. Because of this head and the use of reciprocating pumps, it was extremely difficult not to exceed desired pressures at the top of the grout pipe. Several methods of pressure relief or automatic pressure control were considered. The contractor suggested recirculating the grout through a Moyno pump located on the grouting platform. This was tried and found to be satisfactory. Rates of pumping and pressures could be controlled precisely. Also, the smooth, pulse-free pumping of the grout apparently contributed materially to the ease with which the grout could be injected and reduced premature blocking of the grout holes.

Grouting.—While grouting of the two test areas was being completed, drilling and grouting of the holes near the dam abutment were started at a hole spacing of 12 ft. Test wells were driven in the two test areas on their completion and pumps were installed. The wells were tested both in inflow and outflow. These tests showed that, while a substantial reduction in permeability had been achieved, the overall permeability was still too great.

The work to date was carefully reviewed and after some further experimentation the following procedures were adopted:

1. A maximum hole spacing of 12 ft along the row.
2. To assure sequential grouting of the less pervious materials and to prevent displacement of partially hardened grout, the sequence shown on Fig. 8 was used.
3. A quantity injected or "take" of 50 cu ft to 60 cu ft of grout per linear foot of hole was desired where possible in the outside rows. To achieve this grouting to a take of 40 cu ft per lin ft was with the thinnest mix, 4.0 to 1 water-cement (w/c) ratio by volume, then at 3.0:1 w/c ratio to 60 cu ft per lin ft, and thereafter at 2:0:1 w/c ratio. Grouting was to refusal. For the center line of holes, only the grout mixes thinner than 3.0 to 1 w/c ratio were used and grouting was to refusal without limitation on the quantity pumped. Subsequently, it was found desirable to limit takes in the outer rows to 120 cu ft per lin ft of hole in a few zones.
4. Heave of about 2 in. maximum had developed using pressures originally specified. Pressures were limited to 30 psi measured at the casing head since computation and experiment showed no difficulties at this pressure. This gave an overpressure in the formation of about 75 psi. A continuous system of observing for heave was established.

A review of detailed grouting records indicated there were a number of zones which had accepted much less grout than had been anticipated or was considered necessary to assure satisfactory results. Observations showed the grout to contain substantially larger amounts of material coarser than 200 mesh than had originally been present. It is believed that these coarse particles came from three sources. First, there was an increase in the clay deposit of the proportions of silt and fine sand. Secondly, zones were encountered in the clay deposit which contained an appreciable amount of small concretions. Large concretions were removed in the screening process. Small concretions, however, passed through the screen and were broken up in the mixing and pumping operations. Third, apparently there was contamination of the clay dumped at the plant either by wind or by materials tracked onto the clay during operations.

To avoid these difficulties and improve grout acceptance, use of the clay was discontinued and bentontie substituted. Processed Wyoming bentonite of 200 mesh size was selected and used, since experiments indicated that this material gave the best results and that the fine grind assured a minimum of materials coarser than 200 mesh. The final mixtures developed contained about 5% to 5-1/2% of bentonite to weight of cement. This content was adjusted from time to time, as necessary, using as a criterion that the maximum sedimentation for the thinnest mix should not exceed 2% by the time the mix had gelled. Water-cement ratios by volume were varied in the range from 2:1 for the thickest to 4:1 for the thinnest grouts. Later this was reduced to a maximum of 3.5:1 for the thinnest grout.

The cement-bentonite was smooth and plastic. It contained almost no material of sufficient size to be felt as grit. Sieve tests showed less than 1% of solids coarser than 200 mesh in the fluid grout ready for pumping. Cylinders cast by injecting grout into submerged 3/4 in. gravel showed compressive strength after submersion for 28 days of about 150 psi and coefficients of permeability of about 1×10^{-5} cm per sec.

Immediate improvement was noted with this type of grout. Quantities injected increased to more desirable and uniform amounts.

The three center rows of holes in the abutment area, which had previously been grouted with cement-clay, were redrilled and regrouted with cement-bentonite. Additional holes were drilled through the two test areas and in the abutment area. Substantial quantities of grout were injected into a number of these holes.

The cement in use had a Blaine specific surface ranging from 3,200 sq cm to 3,400 sq cm per g. It contained approximately 1% to 2% of material coarser than 200 mesh. An experiment was run to see whether using finer cement would give significantly better results. Type III cement, having a Blaine specific surface of approximately 4,500 sq cm per g and containing 1/2% to 1% of material coarser than 200 mesh, was tried. No significant difference in the quantity of grout injected could be determined. Difficulty was experienced with this grout setting up in the grout lines. It was concluded that the additional cost of using this more finely ground cement, which amounted to about $0.20 per bag, was not warranted.

Despite the improvements noted with the change to bentonite, review of the data indicated that sand members of significant extent still remained ungrouted. These could be grouted only by using chemical grouts and a detailed review of chemical grouting materials and procedures was made.

Experiments showed that, when mixed with the clays and cements in use at the site, sodium silicate-sodium aluminate mixtures would not gel. This apparently was contrary to most previous experience with these materials, since the difficulty is usually premature gelling. Experience of others has shown that control of gelling of silicate grouts is difficult, and apparently minor changes in ground water chemistry or pH can cause erratic gel times.

Samples of ground water recovered from the wells in the test patterns showed high concentrations of dissolved salts and pH values in excess of 11.0. Erratic variations in these conditions might be expected depending on local permeabilities of the formation following grouting with cement-bentonite.

Under these conditions, difficulty was anticipated with controlling gel times of silicate grouts, and there was considerable question of whether gelling of sodium silicate-sodium aluminate mixtures could be assured.

Chrome-lignin has been used to some extent as a chemical grout. It is a true solution and of low viscosity. The stabilized material is weak and the long-time stability is questionable. For these reasons it was rejected.

A new chemical grout known as AM-9 had become commercially available. In this process, an acrylimide monomer in water solution and a suitable catalyst are injected in a single injection process. Mixing of the catalyst and monomer is done through a mixing valve at the top of the grout pipe. By regulating relative quantities of monomer and catalyst, gel times can be controlled accurately and may be varied at will. By selection of proper catalysts, the process can be made relatively insensitive to variations in pH, especially in the basic region, or to reasonable changes in ground water chemistry. As injected, it is extremely fluid, having a viscosity of approximately 1.7 centipoise. The resulting gel is insoluble and is believed permanent. The gel itself apparently is impermeable, at least within the range of conventional soil testing procedures. It is not subject to synaeresis, nor to changes in moisture content with pressure, and stabilized material has adequate strength and toughness. It is not thixotropic, nor will it heal once it has been ruptured.

Studies indicated that this material, while relatively expensive, would have substantial advantages in ease of placement as compared with silicate gels and probably would be competitive cost-wise on the overall project. Accordingly, a test program was initiated which showed:

1. Pumping and mixing equipment developed by the manufacturer was satisfactory and could be operated in the field to give proper mixing and control of gel times.
2. The material would gel properly if injected into the soil under the conditions present in the grouted cutoff.
3. Even a single row of holes of chemical grout injected within the cement grout curtain area would cause a substantial further decrease in permeability.

On the basis of these tests, it was decided to proceed with the use of AM-9 chemical grout using ammonium persulfate as the catalyst for the final sealing of sandy zones in the grouted cutoff.

Initially, the following principal criteria were established for chemical grouting:

1. The records of adjacent cement-bentonite holes would be reviewed. Where the quantity of cement-bentonite injected at adjacent holes at the same elevation was sufficient to assure that all more permeable strata had been grouted with cement-bentonite, a single line of chemical grout holes would be

used located along the center line of the grout curtain. Where the quantity of cement-bentonite injected was limited, three rows of chemical grout holes would be used; a row along the center line of the grout curtain, a row about 6 ft upstream of this and an intermediate midway between to be grouted last.

2. Primary holes would be located along the center line of the curtain midway between the cement-bentonite holes. Secondary holes would be located midway between primary holes and 2 ft upstream of the center row of cement grout holes.

3. Criteria were established in the form of curves setting up the quantity of chemical grout to be injected in any zone, based on the average quantity of cement grout injected in adjacent holes at the same elevation.

4. If extremely permeable zones were found, additional holes would be drilled and grouted at locations established by the engineers to assure sealing.

5. Drilling and grouting would be by zones from the top downward, using the same basic techniques as for the cement-bentonite grouting.

6. After thorough cleaning of each zone, it would be water tested to determine the coefficient of permeability. If the coefficient was less than 300×10^{-4} fpm, chemical grout would be injected; if more, cement grout would be used. Computation of the coefficient was based on the assumption that flow away from the hole would be through 360° but without vertical spreading.

7. Gel times would be set to permit injecting the estimated total quantity of grout just before the first grout injected begins to gel.

8. Where tests indicated low permeability, a stage length of as much as 15 ft could be used.

Chemical grouting was started on February 10, 1959, beginning at the abutment and proceeding eastward. This work was started on the basis of these criteria and was limited to the primary holes only, except that additional holes were drilled and grouted adjacent to a few holes which showed abnormally high permeabilities and which required cement-bentonite grouting.

Drilling for chemical grouting was done principally with rotary equipment and water since experience showed there was not sufficient flow of water available within the grouted area to provide a satisfactory air lift effect for use of the percussion drills.

The mixing plants were purchased from the AM-9 manufacturer which had developed them. Services of an engineer to set up these plants and train personnel in their operation were also supplied. The flow diagram of the mixing plant is shown on Fig. 9. The pumps supplying both the AM-9 solution and catalyst solution were controlled by the same circuits which were provided with a pressure actuated limit switch to prevent excessive grouting pressures. Outputs of both pumping systems were continuously variable throughout their entire range, permitting accurate control of pumping rates and gel times. The several units were cross-connected with a water tank which facilitated flushing equipment and drill holes.

The work was reviewed from time to time by the board of consultants for the project. They recommended that rather than an arbitrary limit on the quantity of grout, all holes be grouted to refusal at 50 psi overpressure and 3 gpm rate using chemical grout and that grouting be continued on intermediate spa-

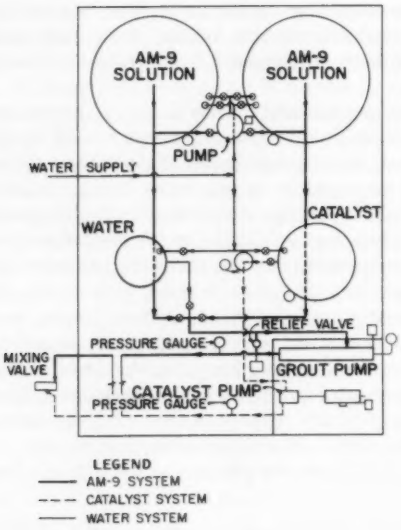
AM-9 SOLUTION
AM-9 SOLUTION
PUMP
WATER SUPPLY
WATER
CATALYST
RELIEF VALVE
PRESSURE GAUGE
MIXING VALVE
GROUT PUMP
CATALYST PUMP
PRESSURE GAUGE
LEGEND
AM-9 SYSTEM
CATALYST SYSTEM
WATER SYSTEM

FIG. 9

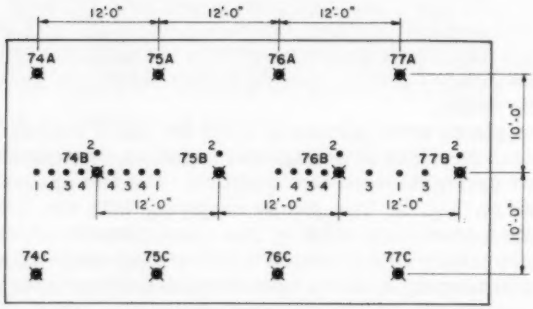
12'-0"
12'-0"
12'-0"
74A
75A
76A
77A
10'-0"
74B 2
75B 2
76B 2
77B 2
1 4 3 4 4 3 4 1
1 4 3 4 3 1 3
12'-0"
12'-0"
12'-0"
10'-0"
74C
75C
76C
77C
CEMENT GROUT HOLE
CHEMICAL GROUT HOLE
NUMBERS SHOW SEQUENCE OF DRILLING & GROUTING CHEMICAL GROUT HOLES

FIG. 10

cings of chemical grout holes until the following maximum takes were not exceeded:

1. To a distance of 200 ft from the abutment, an average of 5 gal per ft of hole.
2. From 200 ft from the above abutment to row 104, 1,200 ft from the abutment where the cutoff crossed the bedrock high, an average take of 7.5 gal per ft, excluding a single horizon of large take if present, but no single horizon of 5 ft height to exceed 40 gal per ft.
3. Beyond row 104, chemical grouting would not be required.

Grouting was completed in accordance with these recommendations. From a detailed study of the results of both cement-bentonite and primary chemical grout holes, several areas were selected where three rows of chemical grout holes were required. The average take in the final check holes was 2.8 gal per ft of water-thin chemical grout. Typical final hole locations through the

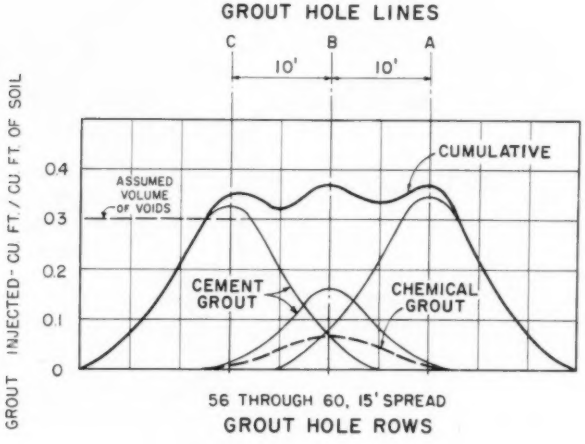

FIG. 11

body of the cutoff are shown on Fig. 10. Final spacing of holes along the center line out to row 104 varied from 18 in. to 6 ft with a substantial portion of the area grouted at 3 ft spacing. Hole spacing in the single line portion of the cutoff across the secondary channel varied from 6 ft to 12 ft. The cutoff terminated at row 167, 1,850 ft from the abutment.

EVALUATION OF RESULTS

If it is assumed that the fraction of voids filled with grout at any distance from a grout hole follows a Gaussian distribution, graphs can be prepared showing the cumulative distribution of grout through the soil mass. A cumulative curve for a typical area is shown on Fig. 11. Similar graphs were prepared for a number of areas along the cutoff. Assuming the average porosity

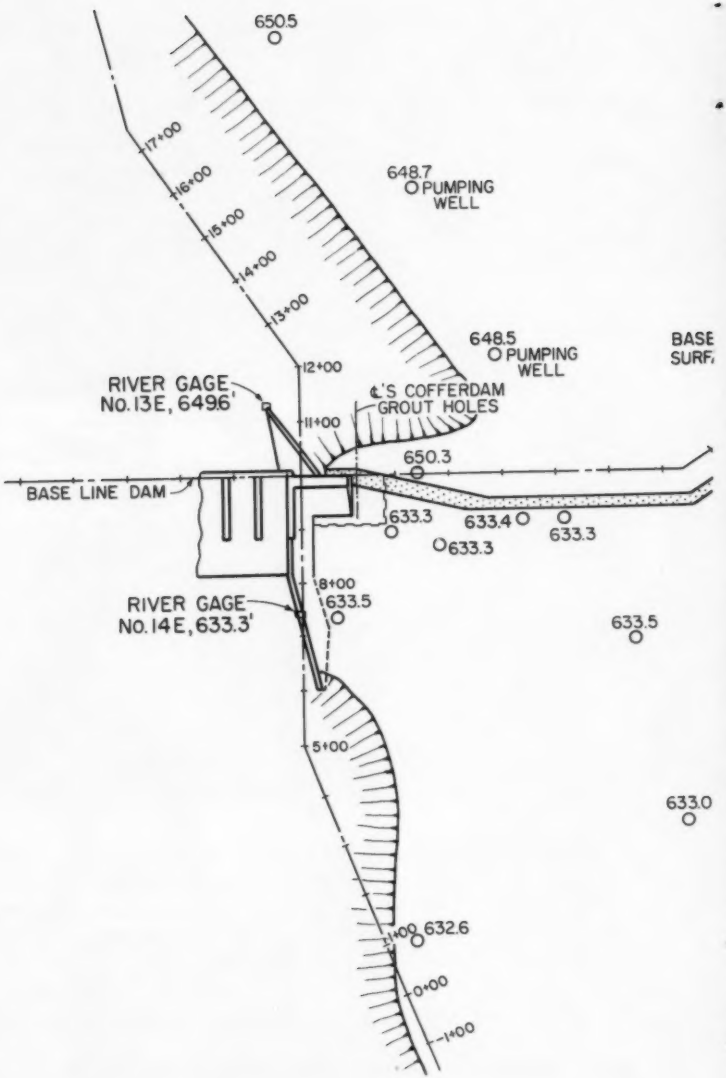
650.5
17+00
16+00
15+00
14+00
13+00
648.7
PUMPING WELL
648.5
PUMPING WELL
BASE SURF.
12+00
RIVER GAGE NO. 13E, 649.6'
℄'S COFFERDAM GROUT HOLES
11+00
650.3
BASE LINE DAM
633.3
633.4
633.3
633.3
8+00
RIVER GAGE NO. 14E, 633.3'
633.5
633.5
5+00
633.0
632.6
0+00
-1+00

FIG. 12

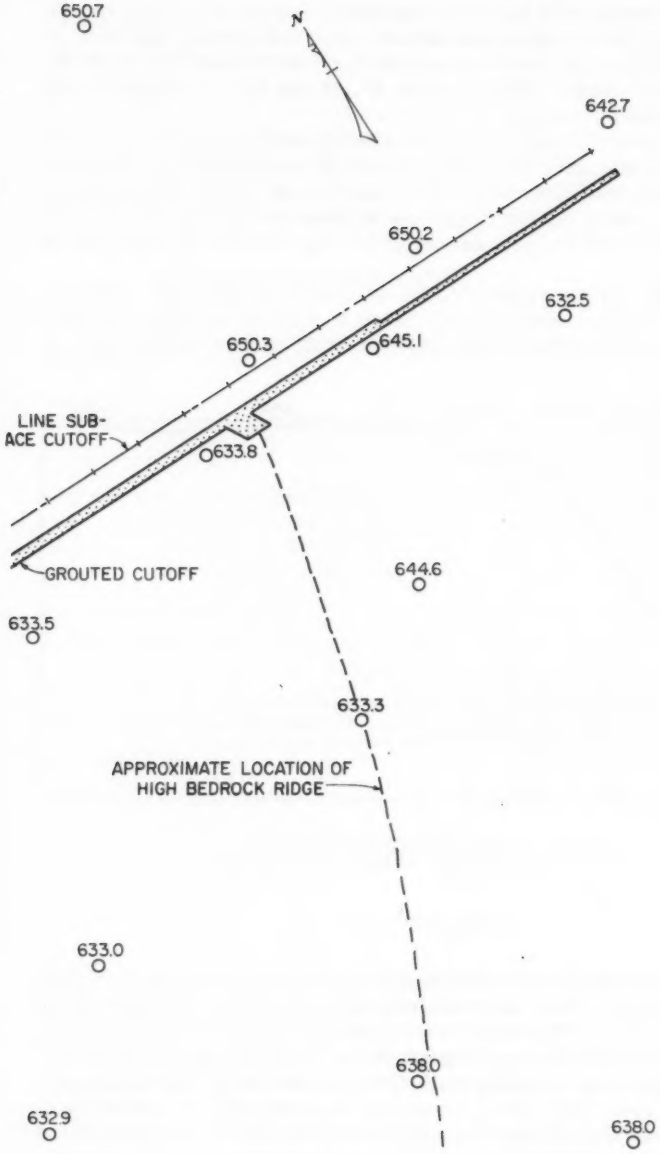

FIG. 12.—*CONTINUED*

of the soil mass to be about 30%, these studies indicated that the desired width for the cutoff of 30 ft was achieved.

The most direct measure of the effectiveness of the cutoff is the relative head difference across the cutoff measured upstream and downstream of it as compared with the drop in the river across the dam, which was 15 ft to 17 ft, depending on the river stage. This is about 1/6 of the final maximum head which will be developed by the dam.

Most of the exploratory borings were converted to piezometers to measure water levels in the lower gravels. Also, additional temporary piezometers were installed immediately upstream and downstream of the cutoff and three wells upstream were used. River levels were measured at four points, respectively, 120 ft and 2,450 ft upstream and 260 ft and 1,650 ft downstream of the face of the dam.

Piezometer levels and corresponding river levels on June 26, 1959 are shown on Fig. 12. The date was selected since it was the crest of the 1959 flood with maximum river differentials, although final check grouting was not

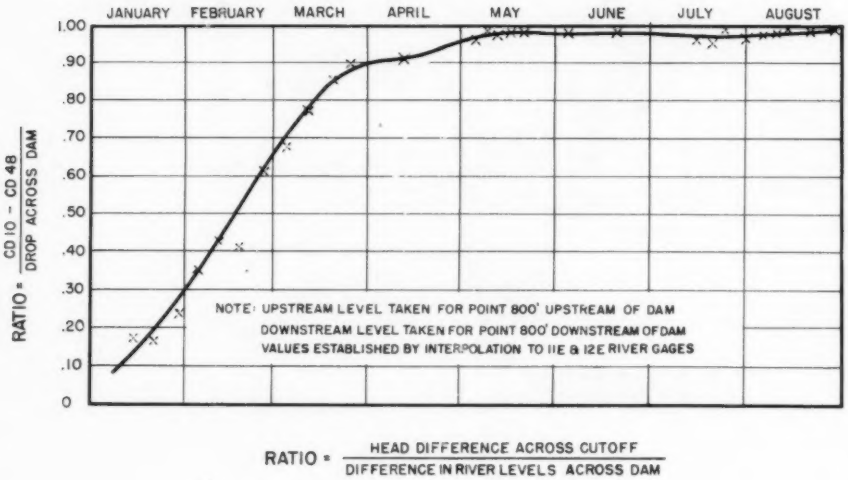

FIG. 13

complete. Upstream of the dam the piezometer readings were essentially level east to west and showed a slope upstream slightly flatter than the slope of the water surface in the river. The water level immediately upstream of the cutoff was slightly higher than the water level in the river measured at the upstream end of the upstream retaining wall. Downstream of the cutoff, piezometer readings were essentially level from east to west, with the slope downstream flatter than the slope of the water surface in the river. The water level in the river at the drain through the downstream retaining wall was slightly higher than the piezometer level in the lower gravels immediately downstream of the cutoff. Accordingly, there was some reverse seepage from the river through the drain and into the lower gravels.

The development of head difference across the cutoff as the work progressed is shown on Fig. 13, in which the drop of water level from piezometers

CD-10 to CD-48, respectively, upstream and downstream of the cutoff, is plotted as a percentage of the difference in water elevations for a point approximately 800 ft upstream of the dam and a point approximately 800 ft downstream of the dam. These points were selected rather arbitrarily because the difference in elevations between river gages 13E and 14E, which are immediately upstream and downstream of the dam, was less than the difference in elevation between the two piezometers. Using these gages would have indicated a cutoff having an effectiveness in excess of 100%. Approximately 90% cutoff had been reached, by late March 1959, almost entirely by cement grouting. As chemical grouting began to be effective, this increased to an indicated overall effectiveness of approximately 99%.

Grouting was completed on August 28, 1959 at direct costs as follows:

Drilling and grouting contract	$1,647,330.74
Chemicals	265,435.88

As a matter of interest, certain quantities of materials used in construction were:

Cement grout	1,068,450 cu ft
Chemical grout	446,241 gal
Casing	95,547 lin ft
Drilling through clay	90,373 lin ft
Drilling through lower gravel	57,274 lin ft
Redrill, grouted zones	142,822 lin ft
Total number of holes:	
Cement grout	554
Chemical grout	527
Quantities of grout injected:	
Cement base - 39.3 cu ft per lin ft of hole	
Chemical - 15 gal per lin ft = 2 cu ft per lin ft of hole	
Materials used:	
Cement	14,534 tons
Clay	1,929 tons
Bentonite	719 tons
Chemicals, dry	127 tons

COMMENTS

While grouted cutoffs have been considered for a number of previous projects in the United States, their actual use in this country is almost without precedent. Inevitably, therefore, this pioneer undertaking will be used as a guide for future projects. Comments and observations on certain phases, which are considered especially important, may therefore, be of significant help to these future developments.

The basic principles which guided the design of the cutoff have been outlined. Of these, the most important is considered assuring sequential grouting in which the more permeable strata, especially those along the outside of the cutoff, are grouted first in order to retain and force grout placed later to enter the less permeable members and seal them off. The spacing and arrangement of holes, the sequence of grouting, the techniques used, and the types of grout used must

be such as to accomplish this purpose. As a corollary, the sequence, spacing and timing should be such that fresh grout will not be placed against partially set grout, because displacement of the partially hardened material may occur.

On the Rocky Reach project, this was accomplished by the grouting sequence, by the time required between stages, and by using two different materials of radically different viscosities to assure that, after the material which could be grouted with cement grout had been grouted, the remaining voids and less permeable zones would be grouted by the chemical grout.

While undoubtedly a substantial reduction in gross permeability and corresponding appreciable head loss can be achieved with a single row of holes, a minimum of three rows is considered necessary where an essentially complete cutoff is to be constructed. Additional rows of holes may be required under adverse conditions or for heads where three rows at design spacing will not give adequate width. For the several reasons enumerated, a minimum curtain width of 25% to 30% of the head on the structure is believed desirable.

The spacing adopted on the Rocky Reach project, both between rows and along rows, required a radial spread of grout of only about one-fourth of the maximum spread observed in the preliminary grouting tests. The basic spacing of holes to assure reasonable success was determined from the two test grout patterns. Holes were added in areas where results were considered poor at the basic spacing. Conditions at Rocky Reach were unusually favorable for grouting because of the thick overlying bed of clay and high permeability which permitted a relatively wide spacing of the holes along the line and a relatively wide spacing of the lines of grout holes. Under less favorable conditions, with greater variations in permeability or lower permeabilities, much closer spacing of individual grout holes and of lines of grout holes probably would be required. Such factors can be determined only by adequate tests and study during the progress of the work.

European experience has indicated that it is not possible to grout completely to the top of an aquifer which extends to ground surface. Some upward restraint is necessary. This may be created by grouting to about 20 ft to 30 ft from the ground surface and then excavating to the top of the grout curtain, or by grouting under some restraining structure such as the clay at Rocky Reach, or under an artificial structure such as a concrete slab. Special care is necessary at the junction of the aquifer with the underlying materials and again at its top. It is extremely difficult to get complete cutoff along these boundaries and careful, thorough workmanship and a program which recognizes these difficulties are essential.

Heave is undesirable and under many conditions may be damaging. The spacing and location of holes should be selected so that the curtain can be constructed at pressures which will not cause heave. Continuous observation to detect and avoid heave is necessary. It is desirable to operate at a pressure as high as can reasonably be tolerated without heaving, yet to avoid the slight overpressure that starts the movement.

The importance of continuous, thorough inspection can not be overemphasized. Continuous supervision was exercised over Rocky Reach work by G. W. Torrance, and Ralph W. Kurtz. Conditions can change radically and erratically within short distances and from stage to stage.

Continuous study is necessary to assure that the best possible results are being achieved. This requires that a complete set of records be kept and that

they be up to date. Fig. 14 shows the data kept for one stage of cement grouting in one hole. Detail sheets summarizing the results obtained in the previous 24 hr were reviewed each morning. In addition, field inspectors were instructed to advise immediately of any unusual or unexpected condition. A continuous running plot was kept, showing the quantities of grout injected in each stage, character of soil and any unusual condition.

Finally, grouting is not a panacea. Used intelligently and with thorough supervision, under conditions and soil characteristics to which it is adapted, it can do a good job, probably at significant savings over many other methods. Complete cutoff of seepage by grouting is improbable, even under favorable conditions. Accordingly, it is essential that proper drainage facilities be provided downstream of a grouted cutoff to discharge safely and, under controlled

CASING

1. Diam.	4"
2. Elev. — Top	666.5
3. Elev. — Bot.	594.1
4. Length	72.4
5. L. F. Recovered	
6. Elev. Ground	
7. Elev. Bot. of Clay	
8. Vol. Backfill	
9. Depth of Hole	127.0

DRILLING

* 1. Elev. Top of Stage	563.7	
2. Elev. Bot. of Grout	544.5	
3. Redrill (1 minus 2)	19.2	ft.
4. Elev. Bot. of Stage	539.5	
** 5. Drill (2 minus 4)	5.0	ft.
6. Time (approx.)	—	

* Same as Grouting Item 5 — Previous Stage.
** Not to exceed 5 feet.

MATERIALS DRILLED	FROM ELEV	TO ELEV
Grout	563.7	544.5
Sandy Gravel	544.5	539.5

GROUTING

1. Packer (Elev.)	661.5	5. Grout Removal (wash) to Elev. (Drilling Item 3 plus 6 ft)	563.7
2. Time (Start)	7.15 AM		
3. Time (Completed)	10.15 AM		
4. Elapsed Time	3 Hrs		

MATERIALS

Batches	Mix	Cem	Clay	Admix	Factor	Volume	Pressure	Time
	3.5:1					81	0	7.15
4	3:1					140.8	0	
14	2:1					352.8	0-30	10.15
							Refusal	
TOTALS								

Vol. in Mixer and Pump			
Vol. in Pipe (length x size factor) =	15	Vol. Mixed	493.6
Vol. in Pipe (length x size factor) =		Vol. Left Previous Hole	81.0
Vol. in Hose (length x size factor) =	36	Vol. Total	574.6
Vol. Wasted		Deduction	51.0

Total Deduction 51.0 Cu. Ft.

VOLUME INJECTED 523.6

NOTE: A brief summary of the drilling and grouting operation must be included on the reverse side of this report.

INSPECTOR H. B.
RELIEVED E. E. W.
RELIEVED BY F. W. J.

Date 12-4-58	Dam Swing	Stage No. 13	Rpt. No. 13	Hole No. 57-C

FIG. 14.—GROUT STAGE REPORT

conditions, the water which seeps through. The cost of these drainage facilities is one of the several costs which must be considered in evaluating the use of a grouted cutoff as compared with other methods at any given site.

CONCLUSIONS

Although final evaluation must await completion and service of the dam over a period of years, it is believed from the piezometric records to date (1961) that an effective cutoff was constructed by grouting these openwork gravels.

From present knowledge, it is believed that the following criteria should govern the construction of such a cutoff:

1. The cutoff must be of sufficient width that the gradients across it are relatively low, a width of 25% to 30% of the head being considered the minimum.
2. Because of the marked differences in permeability which may occur from stratum to stratum, it is essential that small vertical stages be used and the sequence of holes selected be such that the coarser members are grouted first and then the grout forced into the finer, lower permeability members in later stages of grouting. Multiple rows of holes should be used, with the outer rows grouted first and the inner rows grouted last. Final seal grouting with a solution grout of low viscosity rather than a suspension grout probably is necessary to assure grouting of sand members.
3. Grout should be permanent and resistant to either leaching or erosion. Suspension grouts such as cement can be used only for reasonably coarse grained material, an effective grain size between 0.5 mm and 1.0 mm probably being the finest which can be grouted with such materials. Even then it is essential that the percentage of material coarser than 200 mesh be kept as small as possible, preferably, not more than 1% or 2%. Grout must be stable, highly plastic and of low sedimentation.
4. Hole spacings and row spacing must be selected which will assure thorough grouting of the entire width of the cutoff at pressures which will not cause heave.
5. Most important of all grouting is an art. Procedures must be reviewed and revised continuously while the work is in progress. Thorough supervision by competent, experienced personnel who will adjust grout mixes, stages and hole spacings as dictated by experience is essential to achieve satisfactory results.

ACKNOWLEDGMENTS

Rocky Reach dam is being constructed for Public Utility District No. 1 of Chelan County, Wash. Mr. E. C. Metcalf was Chief Engineer for the district; Mr. H. A. Coombs was Consulting Geologist on the project; Mr. R. B. Peck, F. ASCE, and Mr. W. J. Turnbull, F. ASCE served as Consultants to Stone & Webster Engineering Corporation, who designed the project and supervised its construction; and Mr. J. H. Boyd was Project Manager. The Board of Consultants, which was created in accordance with the terms of the bond prospectus, is composed of Messrs. Robert W. Beck, James P. Growdon, F. ASCE, Julian Hinds, Hon. M. ASCE, Samuel B. Morris, F. ASCE, and George R. Strandberg. Sincere thanks are expressed to these gentlemen for their aid and guidance in this work.

APPENDIX.—READING REFERENCES

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E. T. H. Blatter, Research Sta. for Foundation Problems, Report No. 4, Zurich, June, 1941.

2. "Grouting of Foundation Sands and Gravels," Tech. Memorandum No. 3-408, Corps of Engrs., U. S. Army Waterways Experiment Sta., Vicksburg, Miss., June, 1955.

3. "Test of Sanded Grouts: Report 1 - Influence of Chemicals and Mineral Fines on Pumpability; Report 2 - Influence of Sand Grading and Addition of Mineral Fines on Pumpability," Tech. Memorandum 6-419, Corps of Engrs., U. S. Army Waterways Experiment Sta., Vicksburg, Miss., October, 1955.

4. "Chemical Grouting," by T. W. Lambe, et al., Proceedings, ASCE, Vol. 83, No. SM4, November, 1957.

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8. "The Use of Admixtures in Cement Grouts," by Alexander Klein and Milos Polivka, Proceedings, ASCE, Vol. 84, No. SM1, February, 1958.

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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

SYMPOSIUM ON GROUTING: GROUTING IN FLOWING WATER AND STRATIFIED DEPOSITS[a]

By R. H. Karol,[1] M. ASCE and A. M. Swift[2]

FOREWORD

This paper is one of several papers that constitute the Symposium on Grouting that occupied two sessions of the June 1960 meeting of ASCE in Reno, Nevada. Credit for organizing this symposium program goes to a Special Grouting Symposium Committee consisting of Stanley J. Johnson, F. ASCE, Chairman, T. William Lambe, F. ASCE, and W. K. Seaman, F. ASCE.

It is believed that these several papers constitute a significant contribution to engineering practice in the art and science of stabilizing foundation materials by grouting. This is a field in which there appears to be increased interest among foundation engineers and one that is expected to become more important as time goes on and the need to build structures on difficult soil conditions increases.

The papers of the symposium are concerned with cement, clay, and chemical grouting of seams, cavities, and granular materials. The papers illustrate the increased use of chemicals for grouting purposes and also their use with more conventional grouting materials such as clay and cement; grouting of overburdened materials that is rapidly becoming a more definite art and science and that, to some degree at least, is susceptible to laboratory studies; the grouting of a cut-off in granular materials to prevent underseepage beneath a major hydroelectric project (perhaps the first application of this type of grouting in the United States); and the grouting of subsurface voids and solution channels in limestone areas subject to large subsidence.

Note.—Discussion open until September 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings, of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

[a] Presented at the National Meeting of the ASCE, Reno, Nevada, June, 1960.

[1] Supervisor of Soils Engrg. Research, Explosives and Mining Chemicals Dept., Organic Chemicals Div., American Cyanamid Co., Princeton, N. J.

[2] Supervisor of Special Projects, Explosives and Mining Chemicals Dept., Organic Chemicals Div., American Cyanamid Co., New York, N. Y.

Organized in 1952, the Grouting Committee with Task Committees on Cement, Clay, and Chemical Grouting has sponsored groups of papers or symposia occupying the sessions of meetings of the Society in 1954 and 1956, as well as 1960. Papers on grouting that have been published in the Journal of the Soil Mechanics and Foundations Division are as follows: Nos. 1145, 1204, and 1544 to 1552 inclusive. The report of the Task Committee on Chemical Grouting appears as Paper 1426 in the November 1957 Journal. A draft of a report on clay grouting has been prepared by the Task Committee on Clay Grouting; and the report on cement grouting prepared by the Task Committee on Cement Grouting is substantially in its final form.

The Administrative Committee on Grouting takes this opportunity of expressing its appreciation to the task committees for the excellent manner in which they have carried out their assignments and to the individuals who, as authors of papers, have made significant contributions covering the practices of grouting in various fields.

Raymond E. Davis, F. ASCE, Chmn., Administrative Committee on Grouting, Prof. of Civ. Engrg., Emeritus, Univ. of Calif., Berkeley, Calif.

SYNOPSIS

Considerable data have been accumulated since 1956 relative to the effectiveness of various field procedures in grouting. Although the research program has dealt mainly with AM-9 chemical grout, much of the data obtained is believed to be applicable to other solution grouts with comparable viscosities and controllable gel times. It is also thought that the data can be used with more viscous solution grouts, if higher viscosities are accounted for, and with suspended solid grouts provided the solids are not filtered out.

The basic purpose of field grouting is to achieve certain predetermined results. These results are most economically obtained if the size and shape of the mass, stabilized by a single injection, can be accurately predicted. To have a sound basis on which to make such predictions, the effects of a number of variables must be known.

This paper discusses the results of a number of experiments that help to define the effects of ground-water flow and stratification in modifying the shape and size of the stabilized mass that would normally be obtained under static water conditions in uniform deposits. From these results, it has become obvious that the variables encountered in a grouting operation definitely affect the efficiency of an injection, and also that a certain degree of control can be exercised over many of these variables. Several conclusions are postulated.

Since May of 1956, a grouting research program has been carried out, both in the field and in the laboratory, by the manufacturers of AM-9 chemical grout to determine its properties and techniques for its use. This program has turned up much information applicable to all grouts.

The properties of AM-9 have been previously reported.[3] Briefly, it is a mixture of vinyl monomers containing acrylamide and methylenebisacrylamide in proportions that give sufficient cross-linking, when polymerization occurs. The AM-9 is dissolved in water to form a nonviscous solution that will penetrate any formation through which water flows. After the addition of a redox catalyst, the solution is injected or percolated into a soil or rock mass. In a predetermined period of time, that may be closely controlled from a few seconds to several hours by proper choice of catalyst, the AM-9 solution solidifies to a stiff gel. In this manner, any voids are sealed with a rigid, continuous gel that not only prevents the passage of water through the mass, but also binds the particles together and provides a moderate increase in shear strength.

The characteristics of the chemical reaction are such that the viscosity of the AM-9 solution remains essentially that of water until just before the gel forms. With equipment that takes advantage of these properties, successful applications of AM-9 are often made in a fraction of the time and at much lower pressures than required for standard grouting methods.

The low viscosity and controllable setting time of AM-9 make it possible to carry out laboratory scale injections in relatively shallow soil masses, the composition of which can be easily reproduced. This has greatly facilitated a number of studies on basic grouting techniques by making it possible to observe directly the effects of controllable variables on the size and shape of grouted masses.

In order to be able to carry out an ideal grouting program, and given some prior knowledge about the soil or rock mass and the water flow, it would be desirable to know ahead of time what shape and location the grouted mass would take underground. It would be even more desirable to be able to predetermine the shape of the mass and therefore the location of the grout, regardless of the soil and water conditions. Aspects of both these objectives are described in this paper, and simulated combinations of known conditions are used experimentally to evaluate them.

A grouting program or single injection will be considered to be most efficient if the results produced in the ground are the same as those that are desired. For example, Fig. 1 shows two attempts to stabilize a cylindrical mass in a uniform soil. In each case, the same volume of grout was used, but the method used to produce the mass on the left is considered to be more efficient in that it produced more closely a true cylindrical shape. In this instance, the efficiency is related to the distance the pipe is pulled between injections. Many other factors can also effect the precision with which a grouting operation may be carried out.

The major controllable variables affecting the efficiency of an injection are the pumping rate, the setting time of the grout, the pumping time at a given pipe location, the distance-time schedule for pulling or driving the pipe, the grout viscosity, the grouting pattern, and possible inter-relationships between variables. The major uncontrollable variables are ground-water flow and stratification. The effects of the uncontrollable variables were first studied. When these had been determined, tests were made to determine the

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[3] "Development of a New Chemical Grout," by R. H. Karol, Proceedings, American Soc. for Testing Materials, Vol. 57, 1957.

FIG. 1.—EFFICIENCY OF GROUTING IS AFFECTED BY THE DISTANCE THE GROUT PIPE IS PULLED

FIG. 2.—WOODEN TANK USED FOR MAKING SMALL SCALE INJECTIONS

manner in which injection efficiency could be improved by manipulation of the controllable variables.

EQUIPMENT AND MATERIALS

The equipment used to perform all of these experiments consisted of a wooden tank, a pair of metering pumps and a quarter inch stainless steel pipe fitted with a tapered drive point, and with eight holes just above the point. Fig. 2 shows the tank and injection pipe in place. On the ends of the tank are fine screens. The single-acting, variable-speed piston pump, on the far left corner of the tank, pumps water from the rear of the far screen and discharges it behind the near screen. Thus a horizontal flow (simulating moving ground water) can be created.

Two additional pumps, as shown in Fig. 3, are used as a metering system for pumping grout. The larger (volumetrically) of the two handles the AM-9 solution. The smaller handles concentrated catalyst solution. Gel times are adjusted by varying the speed of the smaller pump. Separate discharge hoses from the pumps are brought together in a tee at the top of the injection pipe. This can be seen in Fig. 2.

The material used for performing all of the experiments were clean silica sands with gradation curves as shown in Fig. 4. For reference purposes these are labeled A, B, C, and D in order of decreasing grain size.

INJECTIONS INTO FLOWING WATER

Fig. 5 shows a typical spherical shape made by injecting at a point within a uniform medium sand mass. In this instance, sand B was used. When the permeability is the same in all directions, flow is essentially uniform radially in all dimensions, provided static-head differences are only a small fraction of the injection pressure.

When similar injections are made under conditions of simulated ground-water flow, the major effect of a small flow is to displace the stabilized mass in the direction of flow, without significant change in shape. This is illustrated by Fig. 6. These three injections were made with 5%, 7-1/2%, and 10% AM-9 solutions, respectively, into sand B. A pumping rate of 500 cu cm per min was maintained for 6 min. Gel times were 8 min to 10 min, so that all the grout was placed prior to the start of gelation. The circulating pump was started several minutes prior to the start of pumping grout, and ground-water flow was maintained for 30 min. The flow rate was 480 cu cm per min. Assuming that all the flow took place over a 2 ft-by-2 ft cross section (an area somewhat larger than the screen size), the superficial ground-water velocity was about 1/20 in. per min. Actual velocity would be two and a half to three times this value. Thus, during the induction period (the time lapse between catalysis and gel formation), ground-water moved about an inch, horizontally (0.1 in. per min). This number agrees quite well with the size differential between stabilized masses on each side of the injection pipe. Percentage variations in AM-9 concentration had negligible effects on the experimental results.

As the rate of ground-water flow increases, the effects are magnified. This is illustrated in Fig. 7, that was made in a ground-water flow rate of about an inch per minute. In this experiment, 7-1/2% AM-9, with a gel time

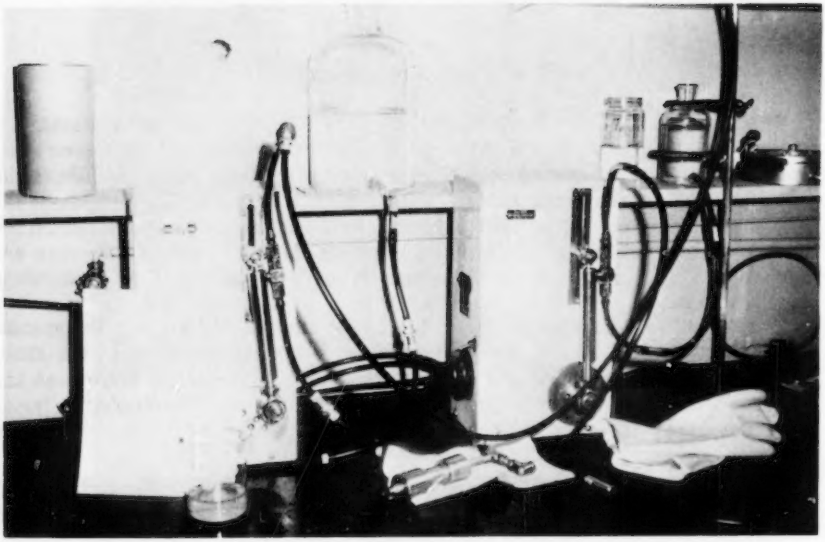

FIG. 3.—METERING PUMPS USED FOR PLACING AM-9 CHEMICAL GROUT

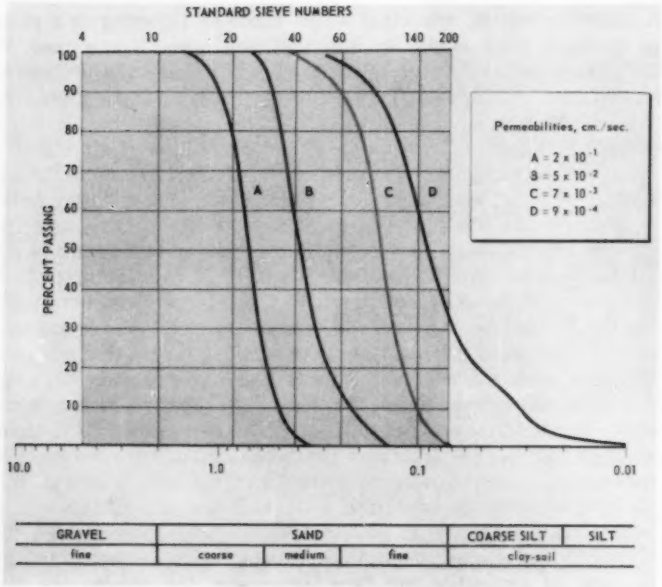

FIG. 4.—GRADATION CURVES FOR THE SANDS USED IN LABORATORY EXPERIMENTS

FIG. 5.—TYPICAL SHAPE FORCED BY POINT INJECTION INTO A UNIFORM SAND MASS

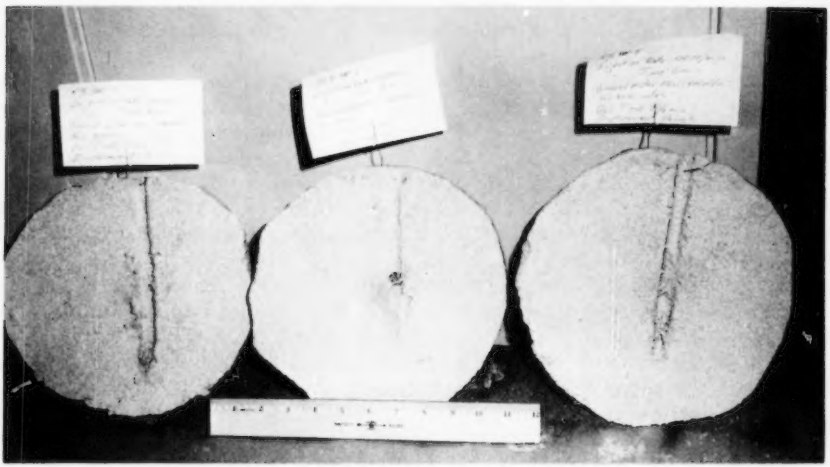

FIG. 6.—DISTORTION OF SPHERICAL SHAPE DUE TO SMALL GROUNDWATER FLOW

of 9-1/4 min, was pumped at a rate of 510 cu cm per min for 6 min. The stabilized mass, excavated at the completion of the experiment, was distorted in the direction of flow and was displaced completely from the injection pipe. In Fig. 7 the pipe is shown in its actual position relative to the stabilized mass.

Previous experiments had indicated that dilution of AM-9 with static ground water was negligible for the prescribed pumping rates. (For example, injections were made into a sand mass saturated with water containing dye. The AM-9 displaced the "ground water" radially, resulting in a stabilized sphere. Wherever dilution occurred, dye would appear in the stabilized mass. In no case has dye appeared in the central portion of a mass stabilized in this fashion. Dye has been found on the periphery of the stabilized masses, particularly at long gel times, such as an hour. The thickness of the dye shell increases with increasing gel time, indicating that diffusion rather than dilution may be the contributing factor. Accurate measurements of the volume of grout and the volume of sand stabilized indicated that loss of grout is negligible for gel times less than 15 min. At longer gel times, stabilized volumes showed an erratic tendency toward decrease, of the order of several percent at gel times of 1 hr.) Visual evidence of the size of the stabilized masses at AM-9 concentrations of from 3.5% to 10% indicated that dilution was negligible for these experiments as well.

To verify this observation additional experiments were performed at ground-water flows of about an inch per minute. For these experiments, a dye responsive to ultraviolet light was added to the ground water in concentrations low enough to have negligible effect on the gel time. The AM-9 solution did not contain dye. Thus, when the stabilized mass was excavated, any areas in which dye appeared were areas in which dilution with ground water had occurred.

Fig. 8 shows a cross section of one of the stabilized shapes made with 10% AM-9 with a gel time of 7-1/2 min. The shades of dye on the downstream side indicate areas where dilution occurred. Pumping rate was 500 cu cm per min for 6 min. Ground-water flow was somewhat faster than that for the mass shown in Fig. 7.

Dilution with ground water occurred only in the downstream side of the mass; in the area in which distortion from the desired spherical shape was acute. The dark shaded area showed a heavy concentration of dye, indicating dilution to an estimated 20% to 50%. The lighter shaded area had a light dye concentration, estimated at less than 20%. Because a 10% AM-9 solution can be diluted with as much as twice its volume of water and still form a gel, it is probable that the stabilized volume accompanying limited dilution will increase, rather than decrease.

Experiments with similar injections, but at much shorter gel times, as discussed later in this paper, indicated that dilution with ground water can be virtually eliminated.

If it is the intention to stabilize the mass of soil surrounding the pipe (as is usually the case), the injection efficiency, shown in Fig. 7, is poor. Moreover, efficiencies can be expected to decrease with increasing rates of ground-water flow. The most obvious methods of increasing the efficiency for a given grout volume would be to increase the pumping rate and decrease the induction time. However, field problems often impose pressure limitations that in turn limit the pumping rates that can be used. The induction period is, of course, always controlled. However, decreasing the gel time without a

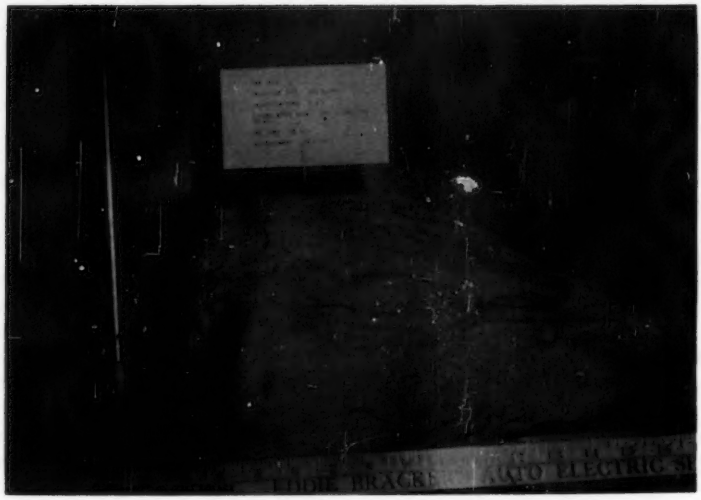

FIG. 7.—DISTORTION OF SPHERICAL SHAPE DUE TO HEAVY GROUNDWATER FLOW

FIG. 8.—CROSS SECTION OF UNIFORM SAND STABILIZED IN FLOWING GROUNDWATER

corresponding increase in pumping rate will often mean that the grout will begin to gel in the ground before the desired volume has been placed. This may, in some instances, interfere with placing the full grout volume.

Initial experiments to evaluate this procedure were done under static ground-water conditions. A typical result is shown in Fig. 9. This experiment was made in sand C, with a gel time of one minute and a pumping time of four minutes. The stabilized mass is roughly spherical in shape. It is thought that the cauliflower shape is due to isolated rupture points made by newly injected grout through incipient gel surfaces of earlier grout. The surface texture and general shape, shown in Fig. 9, is typical for other sands also.

When the short gel, time-long pumping time technique is applied to flowing ground water, the increase in efficiency is remarkable. Fig. 10 shows an injection made under the exact conditions described for Fig. 7, except that the AM-9 concentration was lower (5% versus 7-1/2%) and the gel time was 1 min instead of 9-1/4 min. Laboratory work with AM-9 at other concentrations and in other materials has established that a gel time much less than the total pumping time is a feasible procedure, and that efficiency is greatly increased. Field work has verified this conclusion. Gel times as low as ten seconds have been used on several successful field jobs. It is believed that these jobs would not have been successful at longer gel times.

INJECTIONS INTO STRATIFIED MASSES

Most natural soil deposits will be stratified, and such deposits will have a greater permeability in the horizontal direction than they have vertically. Even a single stratum of soil often exhibits this characteristic. Thus, the idealized shapes that can be obtained by laboratory experimentation under uniform conditions cannot often be obtained in field work. This is illustrated by Fig. 11. This injection was made by slowly pulling the pipe upward, while pumping through strata of A, B, C, and D sands. For this injection, a gel time longer than the total pumping time was used. In a uniform material, a fairly uniform cylindrical stabilized mass would have resulted as shown in the left half of Fig. 1. It is obvious that stratification has seriously reduced the chances of producing a uniform cylinder.

A series of injections is planned into a three-layer system. Each sand stratum will be 4 in. thick. The outer layers will be a medium sand for all injections. The middle layer will use three different sands, ranging down to a fine sand and silt in fineness. By statistical analysis, the variables and their levels are combined so that the separate effects of each may be determined with a minimum number of separate injections.

In theory, a gel time much shorter than the pumping time should help to reduce this problem. The grout injected first should flow into the more permeable zones. When these start gelling, the grout currently being pumped should be forced into the less permeable strata. This procedure will be accompanied by an increase in pumping pressure as the coarser strata are sealed off. These higher pressures, together with the limited soil cover, available in laboratory scale tests, have limited the extent to which this procedure can be verified experimentally. However, the procedure has been

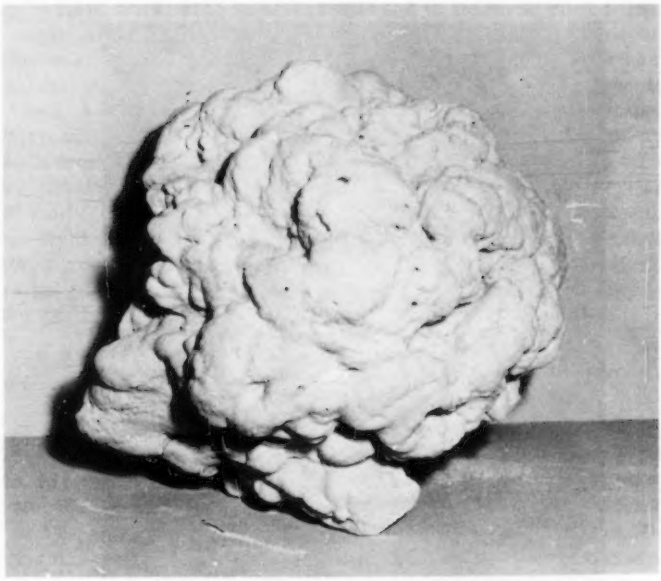

FIG. 9.—MODIFICATION OF TYPICAL SHAPE DUE TO A GEL TIME SHORTER THAN THE TOTAL PUMPING TIME

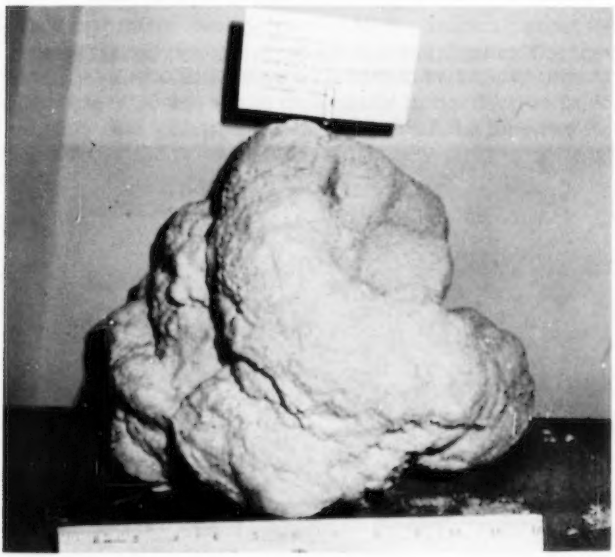

FIG. 10.—EFFECTS OF SHORT GEL TIME USED UNDER CONDITIONS OF FLOWING GROUNDWATER

FIG. 11.—EFFICIENCY OF GROUTING AS AFFECTED BY STRATIFICATION OF THE SOIL PROFILE

FIG. 12.—EFFECTS OF SHORT GEL TIME IN STRATIFIED DEPOSITS

effective in field work, both in soft ground and in fissured rock formations.[4,5] It has also been used successfully in sealing leaks in concrete joints.[6]

Partial verification of the effectiveness of short gel times has been obtained in the laboratory. One such experimental result is shown in Fig. 12. In this case, a gel time of one fourth of the pumping time was used. Visual comparison with Fig. 11 indicates an increase in efficiency in that the grouted mass has a more uniform shape. In theory, the same increase in effectiveness could be obtained with two separate injections, with the first sealing the coarser voids, and the second being forced into the less permeable zones. For large field jobs in which many separate injections must be made to grout the desired ground volume, a proper grouting pattern can therefore do much to insure an effective job. This has been verified many times in the laboratory and in the field.[7]

GROUTING PATTERN IN STRATIFIED SOIL MASS

One of the many laboratory experiments that illustrates the importance of sequence and location of injections was made in an artificial soil profile shown in Fig. 13. The numbers on the left show the depth to each stratum in inches, and the definition of material in each stratum is taken from the information in Fig. 4. The injection pipe was to be driven down to the 20-in. depth and then retracted in steps to the 8-in. depth. The path of the pipe point is shown by the heavy line in the center of the profile. Based on the soil void ratio, 1200 cu cm of grout (100 cu cm per in. of depth) should stabilize a cylinder with an average diameter of 4 in. The vertical dashed lines indicate the average anticipated distance of radial flow. The irregular outline on the profile indicates the pre-grouting estimate of relative penetration caused by permeability differences in the various strata.

For this experiment ten percent AM-9 chemical grout was used with catalyst concentrations set for a gel time of 1-1/2 min. The metering pumps were adjusted to a combined flow rate of 200 cu cm per minute. Pulling the pipe one inch every 30 sec would thus result in the desired injection volume. This meant also that gelation would lag behind the pipe two to three inches vertically. The grouting pattern shown in Fig. 14 was deliberately chosen to illustrate the beneficial effects of intermediate injections (equivalent in the field to grouting to a refusal pressure). In one half of the pattern, only two rows of injections were to be used. It was anticipated that a solid barrier would not be formed in this portion of the grout curtain, because of stratification. In the other half of the pattern, a third intermediate row of injections would be placed to seal the finer strata that the first two rows might not stabilize. The numbers in Fig. 14 near the hole locations indicate the sequence in which a total of twenty-five injections were made. The sequence was chosen so that later injections always fell between two previously gelled areas, a procedure that prior laboratory work had indicated was highly desirable.

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4 "Control of Seepage in Mines," unpublished report of the American Cyanamid Co., October 20, 1958.

5 "Canadian Experiences with AM-9 Polymer," by J. W. McBean and L. J. Cunningham, Ninth Annual Drilling Symposium, Pennsylvania State Univ., October 9, 1959.

6 "Field Work in Vergennes, Vermont," unpublished report of the American Cyanamid Co., June 29, 1959.

7 "Chemical Grout Technique Solves Meramec Shaft Sinking Problem," by Lee Bilheimer, Engineering and Mining Journal, November, 1959.

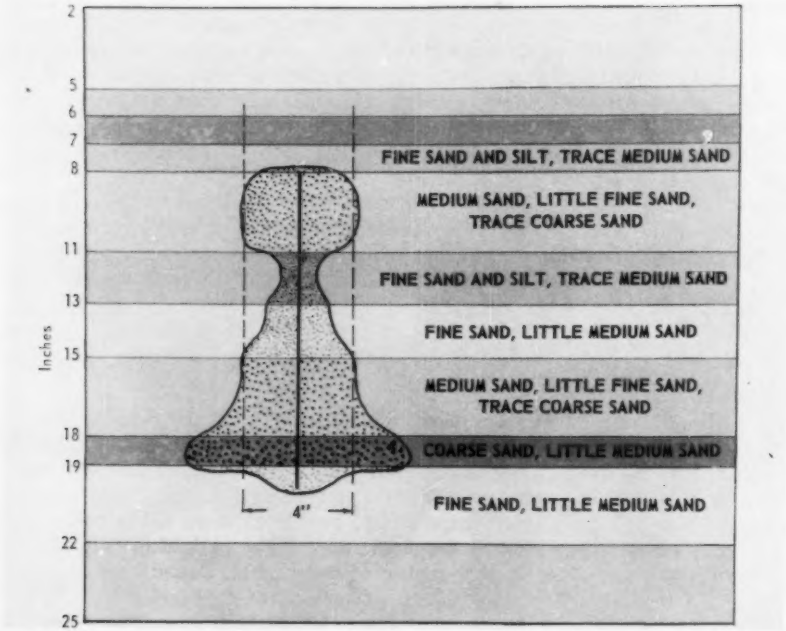

FIG. 13.—SOIL PROFILE FOR EXPERIMENT ILLUSTRATING THE IMPORTANCE OF THE GROUTING PATTERN AND SHOWING THE ANTICIPATED STABILIZED MASS FOR A SINGLE INJECTION

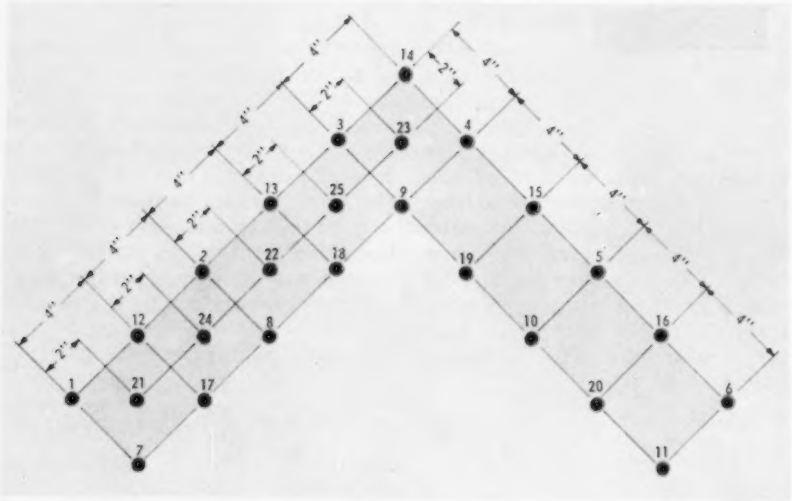

FIG. 14.—GROUTING PATTERN FOR EXPERIMENT IN STRATIFIED DEPOSITS

The grouting program was carried out over a two-day period. Injection pressures for the outside rows were generally of the order of two psi to 4 psi, for shots 1 through 11. For shots 12 through 20, initial breakthrough pressures of 5 psi to 10 psi were common at the start of injection. These higher pressures quickly reduced to 2 psi to 4 psi. In the third row, shots 21 through 25, breakthrough pressures as high as 20 psi were encountered. These generally reduced to 5 psi to 10 psi, shortly after the start of injection. Despite these high pressures, no surface backflow of solution was noted at any time.

After completion of grouting, the unstabilized soil was carefully excavated to reveal the stabilized mass shown in Fig. 15. The individual injections are

FIG. 15.—STABILIZED CURTAIN OBTAINED WITH THE GROUTING PATTERN SHOWN IN FIG. 14, IN THE PROFILE SHOWN IN FIG. 13

clearly delineated, and the average shape of a single injection could be determined quite accurately by measurement. The results of these measurements are shown in Fig. 16. This shows sharper delineation of strata boundries than anticipated (see Fig. 13), and also indicates some mixing of the strata was induced by the vibrations applied to densify the mass prior to injection.

Incomplete stabilization of the fine stratum is clearly shown in Fig. 17, a side view of shot numbers 10, 11, 19, and 20. Even though the major portion of the sand mass is solid, the openings in the fine stratum would render the

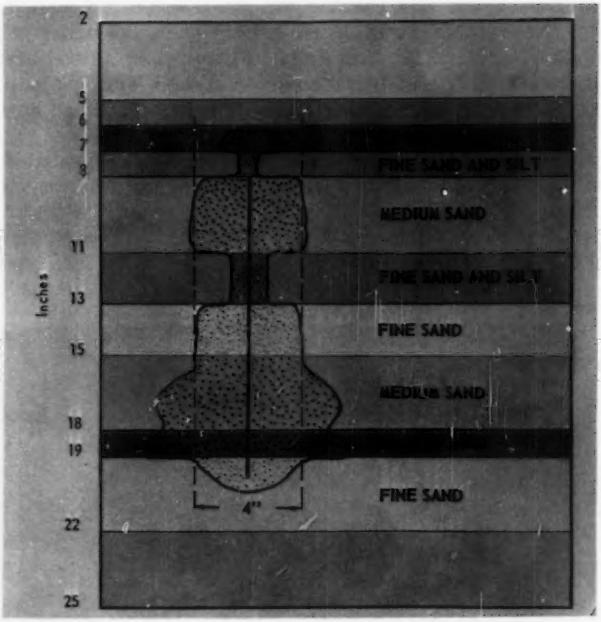

FIG. 16.—ACTUAL STABILIZED SHAPE DUE TO SINGLE INJECTION

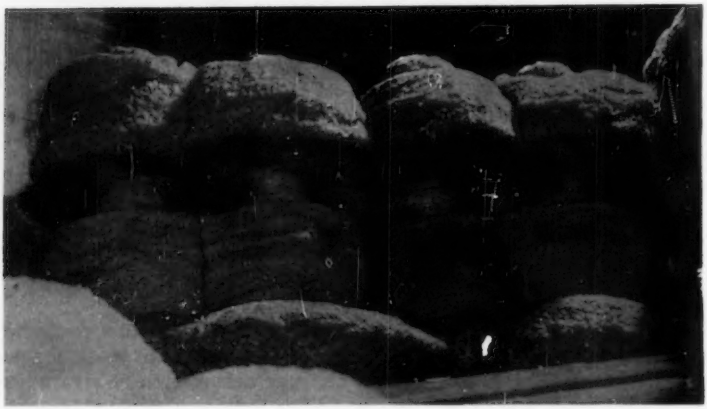

FIG. 17.—INCOMPLETE STABILIZATION CAUSED BY INADEQUATE GROUTING PATTERN

curtain relatively ineffective for prevention of ground-water flow. The improvement in continuity of the grout curtain produced by the third row of shots is illustrated in Fig. 18. Virtually complete stabilization of the mass was obtained, in terms of the formation of a solid stabilized mass in the fine sand and silt layer along the row of injections 21 through 25. It can be seen how these later injections extruded between previous shots, even in the stratum in which overlapping was apparently adequate. This extrusion is also noticeable in Fig. 15.

The results of this and other experiments make it clear that complete and successful stabilization can be accomplished in stratified deposits. It is believed that application of the principle of short gel time with longer pumping time, as previously described, can produce even better results and can minimize the number of injections required. Admittedly, knowledge and control of variables in the field will not be as good as can be obtained in laboratory

FIG. 18.—INCREASE IN GROUTING DEFFICIENCY DUE TO AN ADDITIONAL ROW OF INJECTIONS

work. However, successful work in the field should be possible provided that such work is not done on a hit-or-miss basis.

It is believed that the confining effects of adjacent stabilized masses are important in that they tend to force intermediate injections into the less pervious areas within a soil mass. If this is so, a minimum of three rows of injections is required in stratified deposits. (In uniform deposits, particularly for circular injection patterns, two concentric rows of injections can afford complete stabilization.)[8]

Relatively short gel times should be used, so that gelation is occurring continuously two to three feet below the bottom of the pipe, at all elevations. This procedure eliminates the possibility of pumping all the solution from one injection into one stratum.

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[8] "Shaft Grouting," unpublished report of the American Cyanamid Co., June 20, 1958.

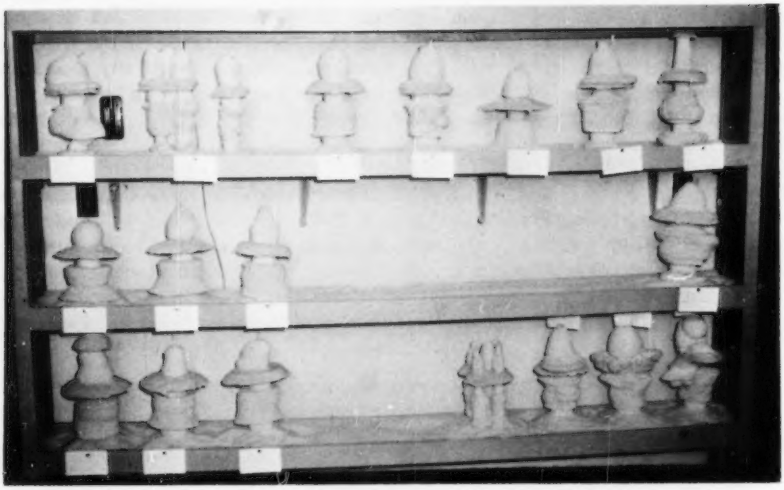

FIG. 19.—RESULTS OF A PILOT STUDY TO DETERMINE THE EFFECTS OF THE CONTROLLABLE VARIABLES ON GROUTING EFFICIENCY

FIG. 20.—EFFECTS OF THE RATIO BETWEEN GEL TIME AND LENGTH OF TIME INJECTION PIPE REMAINS AT EACH ELEVATION

Spacing of injection locations should be based on the average diameter of a stabilized soil column, computed from the volume of the solution to be pumped and the void ratio of the deposit.

A distance slightly less than the average diameter should be used as the basic grid spacing. This spacing should give complete stabilization in the most pervious strata. The final group of injections should be made at the center of each square formed by the basic grid spacing. It should be anticipated that injection pressures for the final injections may be much higher than those required for previous work, and that it may no longer be possible to place any grout in the more pervious strata. Whenever feasible, the sequence of injections should be such that the distance between previous injections is halved.

PRELIMINARY EVALUATION OF CONTROLLABLE GROUTING VARIABLES

The effectiveness of a grouting operation can obviously be affected by variables other than the grouting pattern. Pilot studies have been conducted to determine the most desirable boundary conditions for a comprehensive

TABLE 1.—LIMIT VALUES FOR LABORATORY VARIABLES

Variables (1)	Lower Limit (2)	Upper Limit (3)
Pipe retraction distance, inches	1/2	2
Time between retractions, seconds	30	90
Gel time, seconds	10	110
Viscosity, centipoises	1.6	10

study of the affects of these variables. This larger study has not been concluded. The stabilized shapes resulting from the pilot study are shown in Fig. 19. These have revealed some interesting comparisons. The effects of the ratio of gel time to the length of time the pipe remains in one spot is shown in Fig. 20. These injections were made with the same grout volume through pipes retracted one inch every 15 sec. The stabilized mass on the right was made with a 15-sec gel time, that is, the pipe was retracted at the same time as grout, injected at any one elevation, began a gel. The stabilized mass on the left was made with a gel time of 6 sec. In this case, gelation at every elevation started well before the pipe was retracted. Higher injection efficiency was obtained with gel times shorter than the pulling times in this and in other experiments, and this procedure is recommended for field use in stratified deposits, where pumping pressure limitations permit.

Gel times as short as practical should always be used. Injection efficiency in stratified deposits has been found, in these pilot studies, to decrease with increasing gel time. Fig. 21 shows the result of an injection made under the same conditions as those shown in Fig. 20, except that a gel time of 10 min was used. Penetration is notably less uniform for this long induction period.

AM-9 solution has a viscosity of less than two centipoises, approaching that of water. The effects of viscosity on injection efficiency can be studied

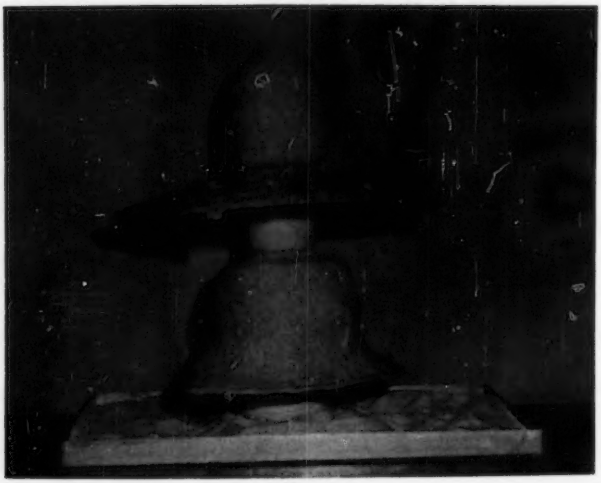

FIG. 21.—DETRIMENTAL EFFECTS OF EXCESSIVELY LONG GEL TIME

FIG. 22.—EFFECTS OF VISCOSITY ON GROUT PENETRATION

by deliberately increasing the viscosity of AM-9 solution with soluble polymers. In Fig. 22 is shown the comparison of two injections made under identical conditions, except that the viscosity of the grout used for the mass on the left was artificially increased to twice that of a normal AM-9 solution. Significantly less penetration was achieved in the finer strata with the higher viscosity grout.

The pilot study has defined limits as practical values for the laboratory equipment to be used. These are shown in Table 1.

CONCLUSIONS

The desired results of a field grouting program are most readily obtained if the size and shape of a grouted mass resulting from a single injection can be accurately predicted. Heterogenous ground stratification and variable ground water flows through such strata drastically modify the results obtained in different locations. Laboratory studies have indicated that several factors are important in minimizing the differences caused by local profile conditions. These factors are related to grout properties, and their use can help in achieving more closely the desired results of a grouting operation.

Most important appears to be the ability of a grout to be pumped into the ground for a period appreciably longer than its setting time. By doing this, the grout gels where it is placed, so that it cannot be carried away by flowing ground water. Also, the first quantities of grout seal the more pervious zones, forcing penetration into the finer zones.

In order to be used in this fashion, a grout must have accurately controllable setting times (particularly in the very short setting times), a low viscosity that remains unchanged until gelation, and a plastic type of end product, so that the still-fluid grout can break through the incipient gel at the leading grout boundaries without fracturing the formation.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SM 2, April, 1961.

A REVIEW AND EVALUATION OF SOIL-CEMENT PAVEMENTS[a]

Closure by James K. Mitchell and Dean R. Freitag

JAMES K. MITCHELL,[1] A. M. ASCE and DEAN R. FREITAG,[2] M. ASCE.—The amplifications on the philosophy and techniques of the testing procedures employed by their respective organizations contributed by Messrs. Catton, Zube, and Maclean and Clare are of particular value. The writers are grateful, too, to Mr. Catton for referring to the work of H. F. Winterkorn, now at Princeton University, whose studies on the physico-chemical aspects of soil stabilization are a landmark in this field. They regret the inadvertent omission of Mr. Winterkorn's papers from their list of references.

The experiences of Maclean and Clare with the stabilization of organic soils in Great Britain will be of particular interest to engineers in the northern United States in which similar problems are encountered frequently. They point out that it is neither organic matter nor acidity alone that indicates a soil will be difficult to stabilize with cement, but rather the simultaneous occurrence of both factors under particular conditions of soil formation. Probably the type and amount of clay minerals present also are of importance. It seems possible that if the physico-chemical relations involved were better understood, these problem soils could be more positively identified and perhaps some means could be developed to permit effective cement stabilization.

E. J. Felt, F. ASCE, raises serious questions concerning the validity of the method of analysis used by the writers and of the conclusions drawn. To answer his objections in detail would require more space than is available here. However, the writers will attempt to deal with the major issues involved.

As Mr. Felt states, it would be desirable to make a comparative evaluation of the performance of various types of pavements by direct test under identical conditions. Unfortunately, this is expensive and there are few such data. The alternative is to compare the results of tests with one type of pavement with the design specifications for equal performance of another type for which established design criteria exist. Any of the several design procedures for flexible pavements could be used for this purpose; but because most of the available test records used the CBR test to evaluate subgrade conditions, the Corps of Engineers flexible pavement design curves were the most logical choice.

Some of Mr. Felt's objections to the analysis may arise from incomplete understanding of the principles on which the Corps of Engineers' flexible pavement design criteria are based. The Corps of Engineers design procedures assume that a pavement may fail as a result of excessive deformations either

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[a] December 1959, by James K. Mitchell and Dean M. Freitag (Proc. Paper 2294).

[1] Asst. Prof. of Civ. Engrg. and Asst. Research Engr., Inst. of Transportation and Traffic Engrg., Univ. of California, Berkeley, Calif.

[2] Chf., Mobility Sect., U. S. Army Engr., Waterways Experiment Sta., Vicksburg, Miss.

in the subgrade or in the base. The thickness specification is intended to provide a sufficient thickness of better-quality material over the subgrade to prevent failure in the subgrade. The thickness requirement does not specify the quality of the material making up the thickness, nor does it make any allowance for reducing the thickness requirement if high-quality materials are used.[3,4] The specifications for quality and compaction in the base course are intended to insure that failures do not occur in the base course. These requirements are not related to the thickness of the base course and do not influence the thickness requirements. Thus, the writers believe that Mr. Felt's objections concerning base course densities, the compaction requirements for airfield pavements as opposed to highways, and his several statements concerning base course failures are not relevant to the thickness analysis, unless, of course, his objection is with the Corps of Engineers design criteria.

Although the Corps of Engineers' flexible pavement specifications do not permit a reduction in the thickness requirement, it must be recalled that these specifications were developed from and intended for use with pavements constructed with uncemented granular materials. There does not seem to be any doubt that a pavement that possesses appreciable flexural strength will not need to be as thick as one that has very little flexural strength. A comparison of the design requirements for a portland cement concrete pavement over a weak subgrade with a similar flexible pavement design illustrates that very well. However, in the case of soil-cement the question is whether its relatively modest flexural strength can, in practice, be relied on to provide a saving in the thickness requirement. The writers concluded from the available data that in general it could not. This is the point with which Mr. Felt has taken issue.

In his discussion of the analysis of the field data, Mr. Felt appears to have placed excessive confidence in the data supporting his position. However, it is the writers belief that the data has been extrapolated beyond the usable range. For example, Mr. Felt infers that at Biggs Airfield 8 in. of soil-cement successfully carried thousands of repetitions of 30-kip wheel loads over a 5 CBR subgrade. Using the Portland Cement Association design curves for rigid pavements,[5] this performance would imply a flexural stress of about 450 psi in the soil-cement. Examination of a table prepared by Felt and M. S. Abrams[6] shows a maximum modulus of rupture of 260 psi for tests on several soil types tested with differing amounts of cement.

Mr. Felt states that the best evaluation of the necessary thickness of soil-cement in comparison to usual thicknesses of flexible pavements is contained in the California Division of Highways design criteria. The writers believe that Mr. Zube's discussion effectively puts this in the proper perspective. Mr. Zube states that their criteria for Class A cement-treated base consider a controlled, uniformly graded aggregate that results in something similar to a low-grade concrete. Thus, the product that the California Division of Highways refers to as Class A cement-treated base is quite different from what most en-

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3 "Airfield Flexible Pavement Design Procedure," T. R. No. 3-475, U.S. Army Engr. Waterways Experiment Sta., Vicksburg, Miss., February, 1958.

4 "Relative Stress Distributing Efficiency of Pavement Layers," M. P. No. 4-84, U. S. Army Engr. Waterways Experiment Sta., Vicksburgh, Miss., April, 1954.

5 "Design of Concrete Airport Pavement," Portland Cement Assoc., Chicago, Ill., 1955.

6 "Strength and Elastic Properties of Compacted Soil-Cement Mixtures," by E. M. Felt and M. S. Abrams, ASTM Special Publications No. 206, 1957.

gineers regard as soil cement. Actually, the material listed in Table 1 of Mr. Zube's discussion as Class C CTB and rated on a par with uncemented base courses probably more nearly merits the name of soil-cement.

In Fig. 1, actual thicknesses of failed soil-cement sections are compared graphically with corresponding flexible pavement design thicknesses. The data for the Brockenhurst-Haddenheim test road in which inadequate stabilization of the organic clay resulted in early failure of the surface have not been included. The dashed line on the plot represents a 1:1 relation, whereas the solid line represents the 1:1.72 relation established by the California Division of

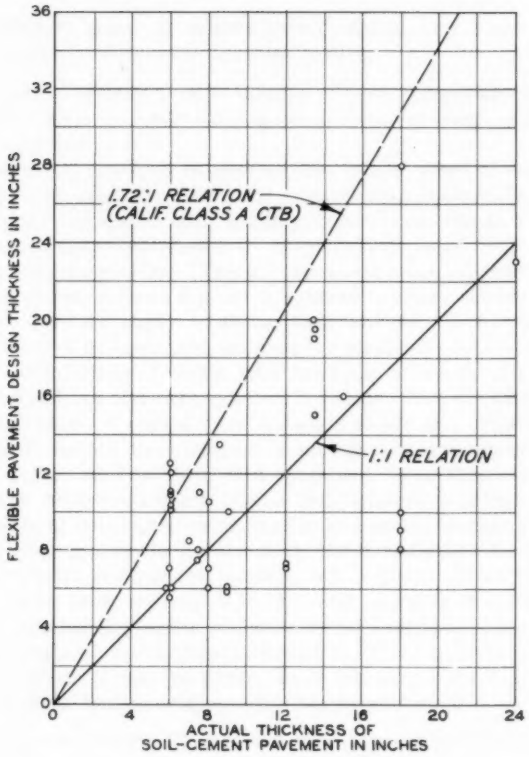

FIG. 1.—FAILED SOIL-CEMENT PAVEMENTS

Highways. The writers believe that most engineers would be reluctant to adopt the California relation for Class A CTB without also adopting the California control specifications.

In conclusion, the writers believe that soil-cement is a very useful engineering material and that it will receive increasing attention in pavement construction. However, in view of the uncertainties in construction control, mixing

problems in fine-grained soils, shrinkage cracking, and so forth, the greatest potential of soil-cement will be to construct base courses that will not be expected to provide a high flexural strength. Furthermore, until additional data become available, they stand on the conclusions presented in the paper. They urge that, whenever possible, complete, well-documented, and factual evaluations should be made of the performance of soil-cement construction (indeed, of all types of stabilized soil). It is only by this means that the capabilities and limitations of this type of pavement construction eventually can be firmly established.

DYNAMIC TESTING OF PAVEMENT[a]

Closure by W. Heukelom and C. R. Foster

W. HEUKELOM,[35] and C. R. FOSTER,[36] F. ASCE.—Mr. Coates raised three specific questions; the writers comments follow:

1. High (plastic) strain occurs mainly at high temperatures under long-sustained loads. Under dynamic loading conditions, however, deformations are almost purely elastic.
2. The duration of the load is comparable in the cases of sustained vibrations and shock-type loading. Thus, the construction materials have the same mechanical properties under both loading conditions. Static loads, however, meet considerably less resistance to deformation, especially on bituminous pavements and clay subgrades. The extrapolated value of R indicates the dynamic stiffness at rates from 8 c per s upwards, where the experiments are carried out, but does not apply to static experiments. The dynamic material constants, introduced in equations derived for statically loaded elastic bodies, proved to give satisfactory results. An example is the good correspondence between wave velocities and extrapolated R values, both of them being related to dynamic conditions. A comparison between static and dynamic properties, for example, CBR values and dynamic E modulus values shows considerable differences (a factor of 5 to 20 in this example). For this reason there is a better agreement between moving traffic and dynamic testing conditions than can ever be obtained between moving traffic and static testing conditions.
3. The vibrating system is governed by the parameters R, M, and A. This applies also to traffic and impact loading, but in these cases the contribution of M and A is much less, so that R is the main parameter. The writers are quite aware that putting the traffic stiffness equal to R is an approximation that, for the present stage of development, suffices. After improving the accuracy of the testing methods it may be desirable to introduce the damping and mass effects for computing traffic stiffness for better accuracy.

Mr. Eremin suggested more illustrative examples. Considerable testing has been done with the vibrator on airfield pavements by the Corps of Engineers. Also, work is continuing in other locations.

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[a] February 1960 by W. Heukelom and C. R. Foster (Proc. Paper 2368).

[35] Konin Klijkel Shell-Laboratorium, Amsterdam (N. V. De Bataafsche Petroleum Naatschappij).

[36] Asst. Chf., Soils Div., U. S. Army Engr. Waterways Experiment Sta., CE, Vicksburg, Miss.

COMPUTER ANALYSIS OF SLOPE STABILITY[a]

Closure by John A. Horn

JOHN A. HORN,[13] A. M. ASCE.—The program and procedures described in the paper mark the furthest advance made by this writer. Unfortunately, the writer has not worked on this problem since August, 1958. However, it is indeed heartening to see now (1961) what great strides have been made by the others still working in this field. Both Mr. B. O. Hardin and Messrs. A. L. Little, N. R. Morgenstern and V. E. Price have significantly advanced the state of the art of "Soil Mechanics Computing" - if it may be called that. Happily, both discussions represent improvements and advances rather than criticisms and disagreements so we are all obviously on the right track. It remains for us to return to our respective computers and start work on the mythical "general/complete slope stability analysis/design" program.

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[a] June 1960 by John A. Horn (Proc. Paper 2501).

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FOUNDATION VIBRATIONS[a]

Discussion by Bobby Ott Hardin, V. A. Smoots, J. F. Stickel, J. A. Fischer, S. D. Wilson, and R. P. Miller

BOBBY OTT HARDIN,[7] A. M. ASCE.—The author has made the theoretical solutions for the various vibratory loadings of a semi-infinite elastic body useful to the practicing civil engineer. As stated in the paper it is necessary to determine the values of the shear modulus and Poisson's ratio of the supporting soil in order to evaluate the dynamic characteristics of a foundation. Ways of evaluating these quantities are listed and some of the soil conditions that will affect them are indicated. The writer is at present carrying out an investigation on the behavior of sands under torsional oscillation. Although only preliminary test results are available at this time (1961) they may be of interest. Some of the results thus far obtained for the effects of pressure and void ratio on the shear modulus of standard Ottawa sand (fraction between No. 20 and No. 30 sieves) are presented here in. The shear modulus was studied by determining the effects of these variables on the velocity of propagation of shear waves through the sand.

Experimental Method.—An apparatus of the electromagnetic type, similar to that used by Birch (54)[2] and others, was used to induce and measure the steady vibration of a cylindrical specimen approximately 12 in. long and 1 1/2 in. in diameter. The specimen was contained in a rubber membrane fastened at the ends to plexiglas end pieces. The specimen and vibration apparatus were placed within a special triaxial cell and air pressure was applied hydrostatically. A wide range oscillator with power supply and amplifier were used to supply the driver with an alternating voltage. The current generated by the pickup was amplified and displayed on the oscilloscope. The resonance method was then used to determine the velocity of the shear wave.

Results.—The pressure range considered was approximately 500 psf to 8000 psf, that corresponds approximately to 5 ft to 80 ft of earth. In one test the pressure was as low as 150 psf and some as high as 9800 psf. Void ratios ranging between 0.51 and 0.63 were obtained. The void ratio was measured with the specimen under a pressure of 13.5 psi. Fig. 16 shows the effect of pressure and void ratio on the shear modulus in dry sands.

For pressures higher than about 2000 psf the shear modulus varies with approximately the 1/2 power of pressure. Below 2000 psf the variation of shear

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a August 1960, by F. E. Richart, Jr. (Proc. Paper 2564).

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2 Numerals in parenthesis refer to corresponding items in the Bibliography.

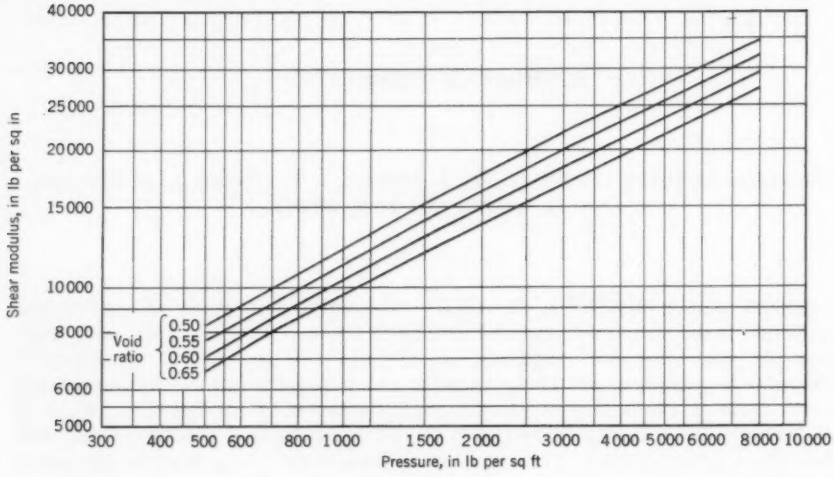

FIG. 16.—VARIATION OF THE SHEAR MODULUS OF OTTAWA SAND WITH CONFINING PRESSURE AND VOID RATIO

modulus with pressure is with the power of the pressure slightly higher than 1/2.

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V. A. SMOOTS,[8] M. ASCE, J. F. STICKEL,[9] and J. A. FISCHER,[10] A. M. ASCE.—The writers will attempt to review portions of the paper in the light of their field experiences. Some of the methods used during the course of their consulting practice will be presented in order to demonstrate the practical applications of Mr. Richart's work.

Probably the initial consideration in the study of vibrations within soils is the determination of the elastic properties of a soil. It is known, however, that soils do not behave in a truly elastic, semi-infinite, isotropic manner and exhibit many non-theoretical characteristics. For example, the consideration of one additional layer in the semi-infinite elastic soil system complicates the problem immeasurably due to the reflection and refraction of elastic waves that occur at any interface. Even in the most simple case, an elastic wave consisting of longitudinal and transverse impulses striking a boundary, theoreti-

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cally no less than twelve new wave types are produced. Further, each wave may generate the Love wave and the Rayleigh wave in the boundary itself (55).[2] In fact it is felt that under certain layered conditions, the most damaging vibrational effects may be due to the reflected and refracted waves due to an underlying denser medium. Consideration should also be given to the fact that both the compressional and shear waves actually contain vectors in three planes.

It is also most probable that even if the foundation soils were closer to an ideal elastic material, the conditions immediately below the foundation would be such that the soils would be stressed beyond their elastic limit.

As mentioned in the paper, the velocity of compression (longitudinal) waves are affected by the presence of ground water in the soil system. If the soils were truly elastic there should be no difference in the elastic properties as long as the soils were in a saturated condition. In general, however, it has been found that the shear wave seems to be little affected by the water table.

Because of these very real problems faced in their work, the writers have felt that the most realistic approach to the problem has been the field measurement of average wave velocities by geophysical prospecting equipment. Even in this method of field measurement, consideration must be given to the type of measurements being made. Because vibratory machinery is not limited to a single degree of freedom in its motion, it is necessary to consider the various modes of vibration that will occur. These modes can include horizontal movements, vertical movements, and rotation about a horizontal axis. Therefore, the writers have attempted, where possible, to determine the elastic properties of the soil in the direction of movement. This has included, aside from ordinary compressional wave velocity measurement, the determination of up-hole velocities and the measurement of surface (or Rayleigh) waves with the resultant conversion to shear wave velocity. These data could be used to determine the shear modulus G (the modulus of rigidity) for use with Mr. Richart's analysis.

The writers have recorded a series of compressional wave velocities of granular soils for a series of different depths at the same sites. A plot of these field measurements on Fig. 6 shows quite a deviation from the suggested relationship between confining pressure and compression wave velocity (Fig. 17). These field measurements were up-hole velocity readings and perhaps may indicate the departure of the soils from an assumed ideal elastic media.

In order to correlate Mr. Richart's suggested procedure with other methods of analyses presently utilized, the writers have performed a series of computations with the data presented as Example A in the paper. The pertinent information is presented as follows: total weight of foundation and engine = 78,370 ib; total unbalanced forces = 4525 lb at 1800 rpm; size of foundation = 17 ft by 4.6 ft in plan dimension (78.2 sq ft) by 16 ft high; soil - sandy clay; modulus of rigidity (G) = 2,500 psi; Poisson's Ratio (μ) = 0.50; and unit weight of soil (γ) = 95 lb per cu ft.

With an analytic procedure using data presented by Gregory P. Tschebotarioff F. ASCE, (56), a resonant frequency for the soil of 622 cycles per min may be determined. Because the operating frequency is 1800 rpm, the compressor foundation should be out of the resonance range as shown in Mr. Richart's example. Tschebotarioff's method is predicated on determining a reduced natural frequency (f_{nr}) and utilizing a series of experience records and is

[2] Numerals in parentheses refer to corresponding items in the Bibliography.

essentially an empirical procedure. First, it is assumed that the natural frequency of resonance (f_n) is proportional to the reduced natural frequency divided by the square root of the bearing pressure (P_o) for different soil types. In this example the bearing pressure is:

$$P_o = \frac{78370}{78.2} = 0.50 \text{ tons per sq ft}$$

The reduced natural frequency from Tschebotarioff's Fig. 1 (56) for a sandy clay soil with a foundation area of 78.2 sq ft is about 440 cycles per min. Because $f_n = f_{nr}/\sqrt{P_o} = 440/\sqrt{.50}$, the resonant frequency of the system is about 622 cycles per min as previously noted.

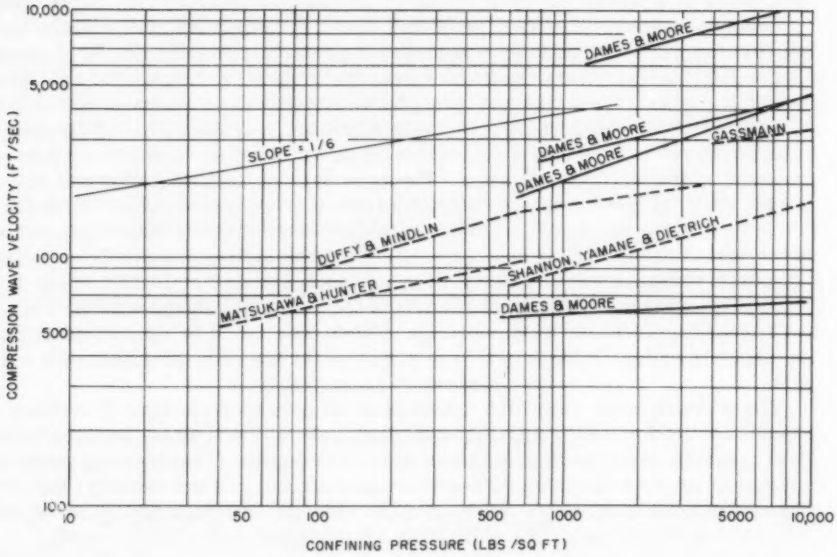

FIG. 17.—EFFECT OF CONFINING PRESSURE ON THE COMPRESSION WAVE VELOCITY IN GRANULAR MATERIALS

If the natural frequency of the soil foundation system, the operating frequency, the weight of the foundation, and the magnitude of the unbalanced forces are all known, then the approximate amplitude of the system may be determined. A method has been suggested by W. K. Newcomb (57), and utilizing this procedure an amplitude of 0.000699 in. has been computed.

Another procedure that is essentially the same as Mr. Richart's was proposed by Isao Toriumi (58). Toriumi integrated the equations of E. Reissner (59) for the case of a uniform foundation load, while Mr. Richart has utilized Mr. Tse Yung Sung's (60) work for the case of a rigid foundation. Toriumi has also assumed that Lame's constants were equal, that is, Poisson's ratio equal to 0.25. While not particularly applicable to Mr. Richart's Example A because

of the rigid base considerations of the engine foundation, Toriumi's method has been used to analyze the same problem in order to develop some comparative answers. For purposes of clarity, the complete mathematical procedure used will not be presented. Toriumi's method utilizes the dimensionless parameters a_o and b as well as an equivalent radius r_o as defined by Mr. Richart. A resonant frequency of 600 cycles per min and an amplitude of 0.000633 in. have been computed using the diagrams presented by Toriumi.

Both the amplitude and frequency values obtained from Tschebotarioff's and Toriumi's procedures agree fairly closely with those found by Mr. Richart's method. In fact the amplitudes computed by the three methods were surprisingly close.

To use Toriumi's work in its present state to determine the resonant frequency of a foundation system requires some additional work. The writers merely approximated a value from Toriumi's graphs and did not carry out the integration process necessary to develop a more precise value. This might have led to the somewhat greater difference in resonant frequencies as computed by the three methods.

Aside from the two methods discussed, the writers have made use of a procedure suggested by Adrian Pauw, F. ASCE, (61) in a series of field tests performed at Kwajalein Atoll in the South Pacific.

This field test was conducted at a site at which the construction of a sensitive radar tower is planned. The radar dish will make fast starts and stops. The impact of these fast starts and stops would cause vibration of the dish due to the elastic flexibility of the mechanical system. Also, the impact would cause vibration of the foundation on the subgrade soils. The foundation soils were saturated, moderately compact sands. If both the foundation and mechanical system should vibrate at the same frequency, they could reinforce each other and seriously delay the ability of the radar to fix on its target.

A test procedure was set up using a large vibrator borrowed from the Navy. This vibrator was taken to the site at Kwajalein and mounted on a large concrete block. After testing on this block, the vibrator was moved and tested on four other foundations of various sizes and weights. This vibrator is capable of developing torsional or tilting forces of 37,000 ft-lb and a vertical oscillating force of 20,000 lb. The weight of the five test blocks varied between about 15,000 lb and 47,000 lb.

Actual movements of the foundation blocks were measured with a mechanical recording seismograph. It has a magnification of both 50 and 100 of actual ground motion and contained three pendulums, one vertical and two horizontal. These pendulums measure the vibration wave in three directions at right angles to each other. The seismograph pendulums had a natural frequency of 1.33 cycles per sec. The vibration machine was operated at a number of different frequencies. Readings were taken at each speed and plotted. By testing enough frequencies, a point of maximum deflection was picked as the natural frequency of vibration for the system.

This frequency of vibration was used to compute backwards and obtain a value for the soil spring modulus as defined by Pauw (61). This soil spring modulus was used to compute the natural period of vibration expected for the large radar foundation installation.

The test blocks were found to have natural frequencies of vibration generally between 15 cycles per sec and 20 cycles per sec. Based on the writers computations, the estimated natural frequency for the proposed radar foundation

is approximately 8 cycles per sec. This frequency of vibration is far enough from the natural frequency of vibration of the mechanical system so that coupling, or resonance, does not appear to be possible.

For purposes of comparison, it is interesting to note the difference between dynamic and static test results. Prior to the vibration tests, a soil spring modulus was estimated using laboratory consolidation and triaxial tests. These tests were run under repeated loading and unloading until a repeatable stress-strain diagram was obtained. Assuming these stress-strain lines to be elastic, a spring modulus was computed for the soil. This modulus was found to be about 1/7 of the modulus obtained from the field vibration tests.

In order to compare Mr. Richart's paper with the writers field test data, a series of computations were made on one of the test foundations using the authors' proposed procedure. The pertinent facts on the installation are: weight of foundation = 26,100 lb; weight of oscillator (W_O) = 5,600 lb; unbalanced rotating mass (M_1) = 3.51 lb- sec^2 per in.; eccentricity (1) = .105 in.; size of Foundation = 6 by 6 ft in plan dimension (36 sq ft) by 5 ft high; soil - moderately compact coral sand; modulus of rigidity (G) - 1075 psi; Poisson's Ratio, assumed (μ) - 0.35; and unit weight of soil (γ) - 120 lb per cu ft. The shear modulus was determined from the soil spring modulus developed in Pauw's analysis and represents an average value over the mass of soil that is assumed to vibrate with the foundation. The measured test values and those computed by the use of Mr. Richart's method are

Measured values - Kwajalein Atoll	Amplitude	0.0022 in.
	Resonant frequency . .	19 cycles per sec
Richart's method of analysis	Amplitude	0.0060 in.
	Resonant frequency . .	9.6 cycles per sec

A portion of the discrepancy may be due to the fact that the modulus of rigidity was determined by the use of Mr. Pauw's method of analysis and, therefore, would not be completely compatible with Mr. Richart's work.

The writers have made other field tests for vibrating equipment. A research-type program was carried out in the vicinity of Martinez, California, on existing compressor foundations and on a proposed compressor foundation, to develop a suitable type of foundation for the proposed compressor. Vibrations were measured at the location of a compressor that had not performed satisfactorily and also at compressors that had performed satisfactorily. Using the measured data, it was concluded that the ordinary approach of changing the dimensions of the compressor foundation would not be as effective and economical as pile foundations.

The scope of the program was to obtain three directional vibration records of frequencies and amplitudes. Four kinds of seismic equipment were used to measure the vibrations. Two of the seismographs were mechanical three pendulum instruments. One had a natural frequency of 1 cycle per sec and a magnification factor of 25. The second instrument had a natural frequency of 1.33 cycles per sec and magnification factors of 100 and 200. The third instrument was a 12-trace refraction-reflection seismograph although only four of the traces were used. Two horizontal linear displacement type detectors having natural frequencies of 5 cycles per sec and two vertical linear displacement type detectors having a natural frequency of 22 cycles per sec were connected to the seismograph to record the four simultaneous traces. The fourth seismograph was an interval time meter for measuring velocity.

The data obtained were used to compute ranges of frequency-amplitude response of the proposed compressor. It was found that testing existing machine foundations provided helpful data for designing adjacent foundations.

There are many practical uses for the data described in the paper and the foregoing discussion, such as:

1. Prevention of transmission of damaging vibrations to adjacent equipment.
2. Prevention of damaging vibrations within the machine-foundation itself.
3. Prevention of settlement due to consolidation, compaction, or lateral displacement of soils.
4. Provision for isolation of vibration sensitive equipment against exterior disturbances.

It should be quite obvious to those who are interested in vibrations and their effect on engineering structures that commercially available equipment is not exactly what is desired at this time. The equipment such as the Sprengnether Seismograph, the Leet Seismograph, and many other three-pendulum units and other single-pendulum low frequency units can only measure a certain range of frequencies, accelerations, and amplitudes. No single piece of gear can actually cover the entire range of needed measurements. It is, therefore, necessary to utilize several pieces of equipment with different ranges and characteristics for the performance of one job.

It appears that the procedure proposed by Mr. Richart will provide a very useful tool in the analysis of foundation vibrations. Naturally, it is necessary to measure the elastic properties of the soil, as well as possible, and use these measured properties judiciously to obtain a reasonable and useful answer.

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S. D. WILSON,[11] F. ASCE, and R. P. MILLER.[12]—The author pointed out, that in order to evaluate the dynamic characteristics of a foundation it is necessary for the engineer to determine (1) the unbalanced forces and operating frequencies of the vibratory machines, and (2) the shear modulus, G, the unit weight, γ, and Poisson's ratio, μ, of the underlying soil.

Test data have been assembled in Fig. 6 demonstrating that the compressional wave velocity for granular materials varies approximately with the sixth root of the confining pressure. Because a wave velocity is proportional to the square root of the corresponding modulus, the modulus should be proportional to the cube root of the confining pressure. This is consistent with an ideal granular medium (that is, theoretical for perfect spheres).

Laboratory data obtained from recent (1961) vibration tests completed by the writers are presented herein. These tests were performed on granular soils compacted with modified Proctor energy, with the exception of the two clean sands that were tamped into the mold by hand.

A vibration apparatus similar to that described and illustrated elsewhere (62)[2] was used in accomplishing the tests. With this apparatus, the specimen is placed in a pressure chamber and the lower end of the specimen is subjected to minute, alternating forced vibrations, at a stress level far below the elastic limit. The resonance frequency is obtained by observing which imposed frequency of vibration gives the greatest output amplitude of a crystal pickup mounted at the upper end of the specimen. The condition of restraint for the specimen being vibrated in this apparatus has been verified to correspond to the lower end clamped, the upper end free.

The apparatus shown in another paper (63) were modified essentially in two respects. The two types of apparatus, one that provides longitudinal vibration, and the other that applies torsional vibration to the soil specimen were combined into one unit; this permits either type of vibration to be applied to the specimen as desired, without resorting to alternating transfer of the specimen between the two pieces of equipment. The advantages of this are obvious. The second modification consists in enclosing the soil specimen within a 6 in. diameter heavy aluminum, air pressure chamber, thus permitting the sample to be tested under various confining pressures. These modifications merely change the configuration but do not change the basic principles of operation of the apparatus (63). (The Nevada samples were tested prior to combining the two apparauts into one unit.)

For the condition of resonance so obtained, the following equation is valid:

$$V = 4 f l = \left(1 - \frac{\pi^2 \mu^2 k^2}{2 l^2}\right) \sqrt{\frac{E g}{\gamma}} \quad \ldots\ldots\ldots \quad (41)$$

in which V is the velocity of wave propagation, in in. per sec; f represents the resonance frequency in cycles per sec; l denotes the length of specimen, in in.; μ is Poisson's ratio; k is equal to the radius of gyration of cross section, in in.; E represents the compression modulus of elasticity, in psi; γ denotes the unit weight, in lb per cu in.; and g is the acceleration of gravity, in in. per sq

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12 Engr., Shannon and Wilson, Seattle, Wash.
2 Numerals in parentheses refer to corresponding items in the Bibliography.

sec. The units are those used herein, however, any consistent system of units may be used with Eq. 41.

Poisson's ratio may be unknown for the soil being tested; but for any assumed value of Poisson's ratio, the maximum error introduced by neglecting the term in parentheses in Eq. 41 will not likely exceed 1% or 2%. Therefore, this term can be neglected and the following equation used:

$$E = \frac{16\, f^2\, l^2\, \gamma}{g} \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

The resonant frequencies at each confining pressure are used in Eq. 42 to compute the corresponding compression modulus of elasticity, E, from longitudinal vibration tests, and the shear modulus of elasticity, G, from the torsional vibration tests. Poisson's ratio may be computed by the following relationship between the compression and shear moduli of elasticity:

$$\mu = \frac{E}{2\,G} - 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (43)$$

The samples reported herein consist in three classifications of granular soils:

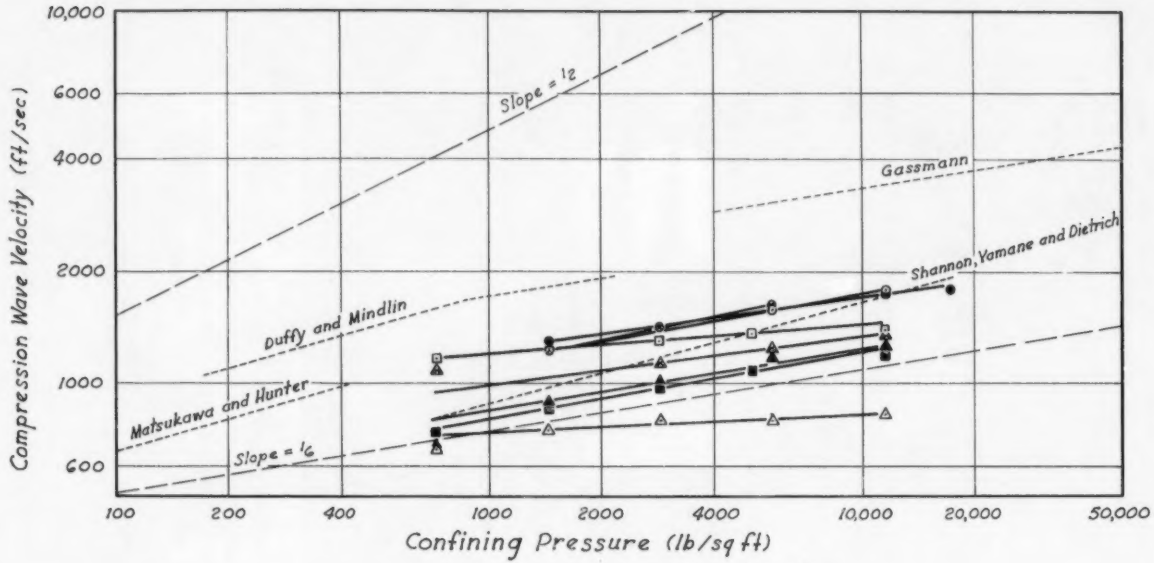
1. Clayey, gravelly sand (SC); three samples from Nevada, and one sample from California,
2. Clean sand (SP); one sample, coarse to medium sand from California, and the other, medium to fine sand from New Jersey, and
3. Silty, gravelly sand (SM) from New Jersey.

These samples are listed on Fig. 18 with the corresponding void ratio, water content, unit weight, and percentage saturation. Though the percentage saturation for sample C-2 (wet) and NJ-2 was computed at the completion of the tests, as given, these two samples were fully saturated prior to testing. Complete drainage was permitted for all samples throughout each test.

The data obtained from these tests have been plotted on log-log graphs, and are presented in Figs. 18, 19(a), and 19(b) that show the effect of confining pressure on the compression wave velocity, on the compression modulus of elasticity, E, and on the shear modulus of elasticity, G, respectively. Fig. 18 is plotted after Fig. 6 and includes some of the data assembled therein.

Samples C-1 and NJ-1 were compacted after removal of the plus 3/4 in. fraction. The reason for the pronounced deviation of these two samples from the general slope defined by the other samples is not definitely known. During testing of NJ-1, however, it was difficult to determine the proper resonant frequency because double peaks were obtained for both longitudinal and torsional vibrations. These two ranges of peak frequencies were nearly parallel in torsion. However, this was not evident for longitudinal vibration, and indicated possible overlapping of the two ranges, particularly at lower confining pressures.

The validity of the test data obtained from C-1 is questionable, though not considered objectionable for inclusion. The specimen size, ratio of length to diameter and general configuration of cap and pickup crystals were considerably different from the other specimens tested. It is also interesting to note that the lower three confining pressures may more correctly define the relationship



			e	w.c.	γ	S_r
Nevada	⊙ YT 1, G-I	(SC) Clayey gravelly sand	0.49	7%	118	37%
	⊛ YT 1, G-1V	(SC) Clayey gravelly sand	0.32	7%	131.5	56%
	● YT 2c, G-II	(SC) Clayey gravelly sand	0.30	8.5%	137	72%
California	△ C-1	(SC) Clayey gravelly sand	0.23	7.8%	147	90%
	▲ C-2(dry)	(SP) Coarse-med. clean grav. sand	0.55	-	108	-
	▲ C-2(wet)	(SP) Coarse-med. clean grav. sand	0.5	7.3%	120	38%
New Jersey	⊡ NJ-1	(SM) Silty gravelly sand	0.34	5.2%	132	43%
	■ NJ-2	(SP) Med.-fine clean sand	0.71	13.2%	110	49%

FIG. 18.—EFFECT OF CONFINING PRESSURE ON THE COMPRESSION WAVE VELOCITY

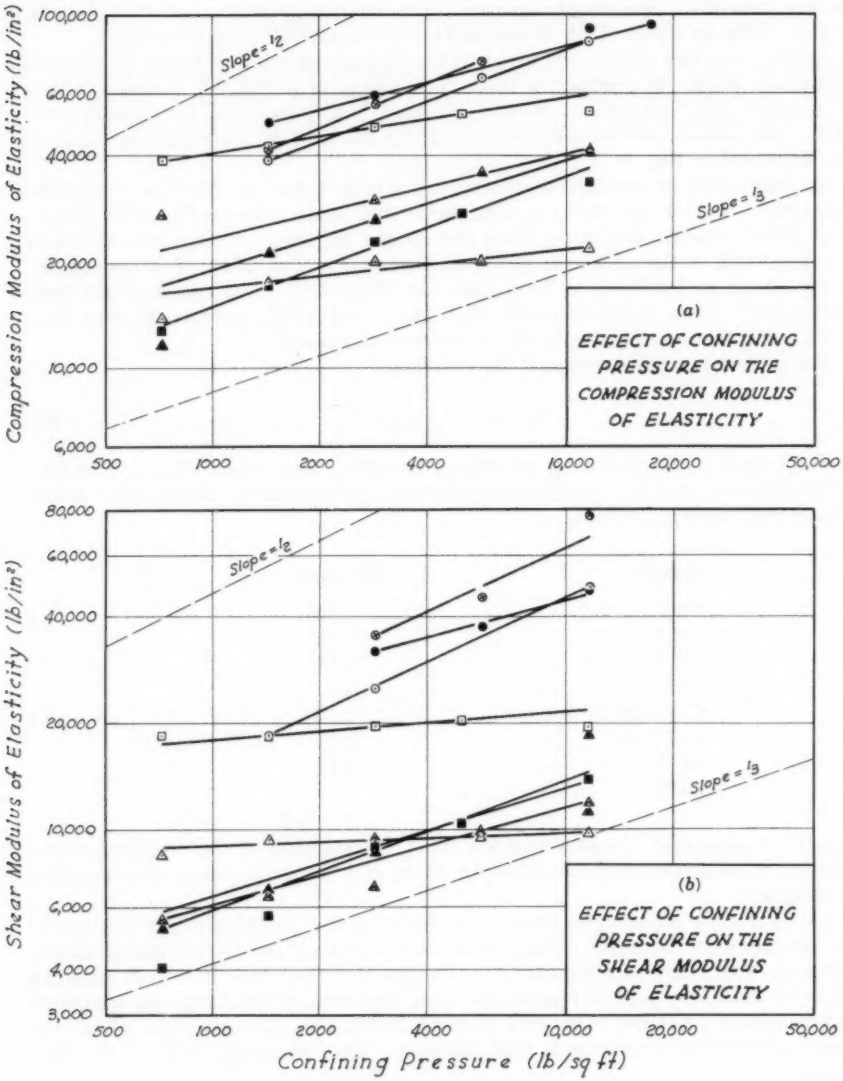


FIG. 19.—EFFECT OF CONFINING PRESSURE ON THE COMPRESSION AND PRESSURE MODULII OF ELASTICITY

for C-1 than the higher three. This would effectively increase the slope of the line. However, this deviation from the line defined by the higher three values may be the result of imperfect contacts at some points of the soil grains under the low confining pressures, or even false frequency peaks due to improper seating of the sample. This deviation is also suggested in C-2 (wet).

Sample C-1 has nearly the same grain size gradation and water content as the Nevada soils. In comparing these four samples in Fig. 18, it appears that the slope of the line defined by each sample decreases with decreasing void ratio (increase in unit weight and percentage saturation) as expected. This is also apparent in Fig. 6.

No conclusion is reached in comparing the data for the clean sand sample C-2, tested in both the oven-dry and saturated state. Generally, E is lower, (Fig. 19(a)), in the saturated state than in the dry state, though this effect is less important as the confining pressure increases. The effect of water content on the shear modulus, G, is less apparent (Fig. 19(b)). It is noted that both "saturated," clean, uniform sands, C-2 (wet) and NJ-2, are nearly superimposed, each having essentially the same moduli values.

The author has indicated that the relation:

$$\text{Modulus} \sim (p_c)^{1/3} \quad \ldots\ldots\ldots\ldots\ldots \quad (44)$$

in which (p_c) is the confining pressure, is valid. Exponents of (p_c) obtained by the writers from the compression modulus curves (Fig. 19(a)), are found to have the following values:

Sample	Exponent
YT 1, G-I	0.37
YT 1, G-IV	0.42
YT 2c, G-II	0.25
C-1	0.1
C-2 (dry)	0.23
C-2 (wet)	0.31
NJ-1	0.13
NJ-2	0.37

With the exception of samples C-1 and NJ-1, the exponent of the confining pressure for all samples approximates the value of 1/3 for the range of pressures investigated.

Data given by Wilson and Dietrich in a discussion to their paper (62) show that the exponent decreases with increasing confining pressure, that is in accordance with Duffy and Mindlin's curve for tests on steel balls. It appears that whereas some granular soils show the exponent of the confining pressure (p_c) to approximate the value of 1/3, given in Eq. 44, other granular soils may show considerable variation in this exponent. No explanation is given for this phenomenon.

It is emphasized, however, that the writers know of no direct correlation of the values reported with field performance of actual structures.

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SEEPAGE REQUIREMENTS OF FILTERS AND PERVIOUS BASES[a]

Discussion by H. J. Gibbs

H. J. GIBBS,[22] M. ASCE.—The author has presented criteria for establishing filter thickness by a practicable and rational method that should be a useful reference for filter design. The interesting feature of the paper is the emphasis on the importance of adequate permeability of the filter and not merely on the requirement of a filter that is more pervious than the protected soil.

Extensive studies in the Bureau of Reclamation for canal and spillway structure design, as shown by K. P. Karpoff, M. ASCE,[23] utilized the maximum flow condition as one of the principal bases for determining limiting gradations of filter material. The coarsest limiting gradation is affected by the ability of the filter to prevent entrance of the protected material into the filter, clogging it and reducing the flow from the maximum that can be attained. On the other hand, the filter must not consist of such a fine graded material that flow is restricted from being the maximum that can be attained. Limitations were established for two gradation values, the 15% size (D_{15}) and the 50% size (D_{50}), for the purpose of guiding the filter gradation curve to be one producing maximum flow. It was believed that when very broad limitations were allowed that an acceptable gradation could be misconstrued such that flow would be restricted and possibly be no better than the material protected. For this reason, flow measurements have always been an important part of every laboratory test made to evaluate the adequacy of a filter gradation selected.

Mr. Cedergren has shown the importance of not only having a filter with greatest permeability that still protects the soil, but knowing what the permeability value of the filter is so that the dimensions of the filter can be established. Therefore, the determination of permeability of the filter would also be an important part of a filter test.

Although the author uses studies of flow nets to establish the design dimensions of the filter, that is a method of study limited by the boundary conditions chosen and generally used only on more important seepage problems, he has converted these studies to simplified charts that may be sufficiently accurate for solutions of many filter problems with reasonably similar boundary conditions. To this end, this paper is a welcome contribution to the profession.

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MECHANICS OF TRIAXIAL TEST FOR SOILS[a]

Discussion by Laurits Bjerrum, Kwan Yee Lo, W. M. Kirkpatrick, A. A. Eremin and Andrew W. Jenike

LAURITS BJERRUM,[3] and KWAN YEE LO.[4]—In order to assure uniqueness of solution, a stress-strain relationship has to be introduced. The one chosen was rigid-plastic together with usual assumptions made in theories of plasticity. It is important, therefore, to note the deviations of actual soil behavior from this idealized case, in particular the invalidity of the two basic assumptions made in plasticity theories when applied to soils. As the author dealt exclusively with sands, the discussion will be limited to granular materials, although some of the remarks that follow apply to clays as well.

The Non-Orthogonality of Plastic Strain Increment Vector and the Yield Surface.—One of the fundamental features of plastic theories, perfect or workhardening, is that the plastic strain, when plotted as a vector in the stress space is normal to the yield surface or failure envelope (27). In the terminology of plasticity, this is known as condition of normality.

Consider the simple case of an element of soil subjected to simple shear as shown in Fig. 14. The quantities σ and τ are the applied compressive and shearing stresses, respectively. In the σ - τ diagram (Fig. 14(b)), the failure envelope for most cases may be approximated by a straight line passing through the origin in the case of granular material. The condition of normality at once requires that the components of the plastic strain increment vector to point in the directions as shown. Because the component Δe_p associated with dilatation is in the direction of tension, expansion in volume during failure is therefore a necessary condition to satisfy the principle of normality. Using this simple procedure to compare the theoretical values with experimental data, the assumption of normality can readily be investigated.

An extensive series of triaxial tests on a fine sand obtained from Trondheim Fjord, Norway, has been carried out and results reported elsewhere (28).[2] These tests include undrained tests with pore water pressure measurements, drained tests, constant volume tests, isotropically and K_o-consolidated over a wide range of porosity. The results of these tests are illustrated in Fig. 15. The values of ϕ both with and without correction for dilatancy (29) are plotted. The most important feature in the diagram is the rapid decrease in the friction angle with high porosities, and failure in compression is accompanied by a volume decrease in this range. Qualitatively therefore, the plastic strain increment vector lies to the right of the normal to the failure envelope as shown

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[a] October 1960 by R. M. Haythornthwaite (Proc. Paper 2625).
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[2] Numerals in parentheses refers to corresponding items in the Bibliography.

by the vector AB for very loose sand. On the other hand, comparison of measured volume increase with shearing strain for samples in the dense range showed that the predicted dilatation far exceeds that observed and the inclination of the plastic strain vector is illustrated by the vector AC in Fig. 14(b). Indeed, the author's test results as contained in Table 2 indicate the same salient feature. The compression tests showed a dilatation much less than the theoretical value, whereas the extension tests showed that the theory again overestimates the contraction.

The Non-Coincidence of the Principal Directions of Stresses and Strains in the Plastic Range.--The second basic assumption made in plastic theories is that De Saint-Venant's principle is valid, namely, the principal directions of stress and strain are coincident. Whereas this principle is valid for plastic materials, there is no obvious physical reason that the same applies to frictional materials that soils are. It has been shown elegantly by Josselin de Jong (30)(31), both theoretically and experimentally and, under plane strain conditions, that in general, deviations of principal directions do actually occur and

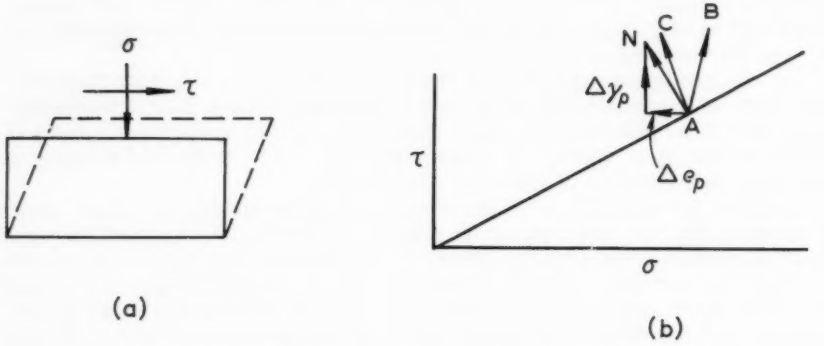

FIG. 14.—ELEMENT OF SOIL SUBJECTED TO SIMPLE SHEAR

the range of the angle of deviation is as great as + ϕ, in which ϕ is the friction angle of the material.

In view of these limitations, it is immediately apparent that theoretical approach to conditions of failure in soils using plasticity theory appears unattractive unless reformulation or radical modification is carried out. This requires the study of the outstanding yet unsolved problem of the stress-strain relationship of soils.

The object of phenomenological approach to study the behaviour of a material at the range of failure is to establish an analytical expression sufficiently general to include the effect of any combination of stresses. Such an expression is usually too complicated for use for practical purposes; but once established, reduction to simple expressions can easily be performed with respect to special conditions. The assumptions and limitations involved in the simplification can readily be perceived. In practice, failure mechanisms generally involve compression or shear. In establishing a failure criterion, therefore, there is both no physical and practical grounds to place undue emphasis on extension

tests whose data are themselves contradicting especially in view of the fact that so far most experimental and field evidence suggests the validity of Coulomb Mohr criterion of failure. Unless and until more data is available especially for stress systems other than those attainable in the triaxial cell, the formulation of a general failure criterion must be considered as premature.

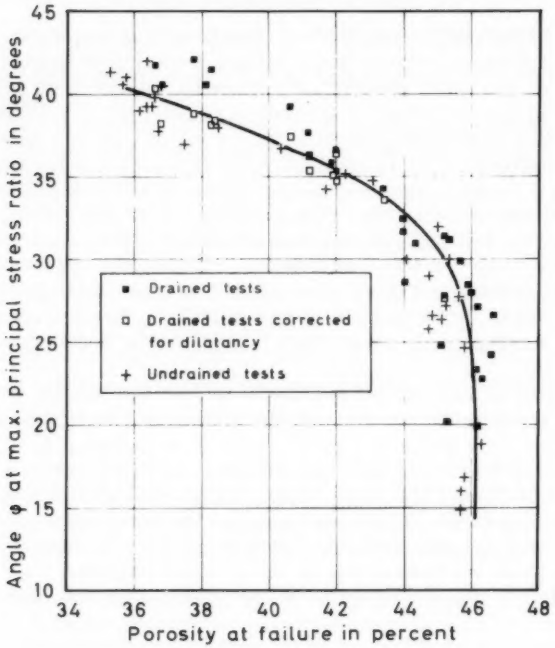

FIG. 15.—POROSITY VERSUS ϕ

The preceding brief comments is not intended as criticism of the author's commendable work. It is only by studying deviations from ideal cases rigorously tested that the actual behaviour of a material can be delineated unambiguously.

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W. M. KIRKPATRICK,[5] A. M. I. C. E.—The question of the influence of the intermediate principal stress on the failure condition of sands is raised in the paper. One of the basic assumptions made by the Coulomb failure theory is that the intermediate principal stress may have any value consistent with its definition without influencing the condition of failure. If this assumption is correct and the failure theory is applicable, the friction angle parameter measured in the triaxial compression test should agree with that found for the different stress configuration obtaining in the axial entension test. The author does not find this agreement, and he refers to results of others including the writer (21)[2] in which agreement was obtained. Before the discrepancies between these quoted results can be cleared up, it must be established if comparison such as that made in the paper is indeed justified. Are in fact the various investigators measuring the same thing in their tests? This does not appear to be entirely so.

In the discussion to the 2nd session of the 4th Conference (32)(33) similar questions were argued, and some of the statements made then are equally applicable here.

Mr. Haythornthwaite, in his tests, assumes failure to occur at an arbitrarily chosen ratio of stress and axial strain. It is emphasized that in the writers tests failure was defined as the peak point on the stress-axial strain curve, that is the point at which the deviator stress reaches a maximum value. Failure in the tests of A. W. Bishop and A. K. G. Eldin (34) was also defined in this way. It is likely that if failure is defined differently the results will also be different.

A modulus, such as that used by the author, provides a means of comparing the stress-axial strain curves. Using the modulus to define a unique state of failure in the axial extension and the compression tests infers that the stress-axial strain curves for the two tests are comparable. The curves, however, express different conditions and are not comparable because in the compression test the axial strain is a strain in the major principal stress direction, whereas in the extension test the axial strain is in the minor principal stress direction. It would, therefore, be surprising if the use of such a modulus could provide results giving equal angles of friction in the extension and compression tests.

The peak point on the stress-axial curve gives a reasonable definition of failure, and its use provides a convenient means of comparing the behavior of samples in triaxial compression and axial extension and also in other tests, such as the thick cylinder tests (21), in which the strain configurations are more complex and are not readily compared with the triaxial strains. The peak point represents a stage in the progress of failure of sands at which the maximum obliquity condition is reached, a stage that is easily recognized. In ad-

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[2] Numerals in parenthesis refers to corresponding items in the Bibliography.

dition, no assumptions are made regarding stress-strain relationships. Mr. Haythornthwaite's results, evaluated at peak point conditions, would be of interest.

It should be pointed out that the writer obtained further confirmation that the intermediate principal stress had no influence on the failure condition, for Loch Aline sand, from additional extension tests not quoted by the author and from the previously mentioned thick cylinder tests. In these latter tests the intermediate principal stress σ_2 had a value approximately equal to $\frac{\sigma_1 + \sigma_3}{2}$, and the results gave angles of friction in close agreement with those found in the triaxial tests.

A yield criterion based on a yield surface of triangular section such as that suggested by Mr. Haythornthwaite is unrealistic in the light of existing knowledge on the strength of sands.

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A. A. EREMIN,[6] M. ASCE.—In the example illustrating the method, Mr. Haythornthwaite considered the plastic state of soil material when no energy of stress-strain is lost. Fig. 11 shows the yield stresses in the crushed quartz sand.

It would be interesting to extend the method of triaxial test by applying it to the analysis of bearing properties of soil material at a loaded structure site. In practice it is often important to know not only the yield limits of the assumed relative axial and transverse pressures, but the maximum bearing stresses with the varied boundary stresses. Note that the boundary stresses vary with changes in the soil moisture content, chemical content of soil, temperature, and magnitude of bearing stresses. Often, the magnitude of boundary stresses may be regulated for the assumed soil properties at the structure site.

In the analysis of bearing stresses and soil stability, it is important to know the factor to safety of the bearing stresses.

The resulting stresses obtained from the triaxial test as developed by the author may be expressed graphically by the Mohr's failure circles from which the cohesive stresses of the soil may be obtained.

ANDREW W. JENIKE.[7]—The writer wishes to interject two thoughts:

(1) As soon as the yielding of a consolidated soil has taken place, the original yield surface at the slip planes is destroyed and is replaced by a new yield surface that is a function of the conditions of stress at the slip planes. Along the slip planes the soil is reconsolidated to a new density.

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Dilatation occurs in the plastic region at the moment of the original failure. Further deformation may involve no density change, or dilatation or, indeed, contraction, depending on whether pressures remain constant, decrease or increase.

This explains the low experimental values of dilatation as compared with the predictions based on continued dilatation at the original yield surface.

(2) For cohesive sands, the angle of internal friction ϕ is not constant but varies to a maximum that is probably 90° at the intercept of the yield locus with the σ axis in σ - τ coordinates. This point has been discussed elsewhere.

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